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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

VOLUME VII. 1912.

Published at North British and Mercantile Building,
Waterloo Place, London, S.W.

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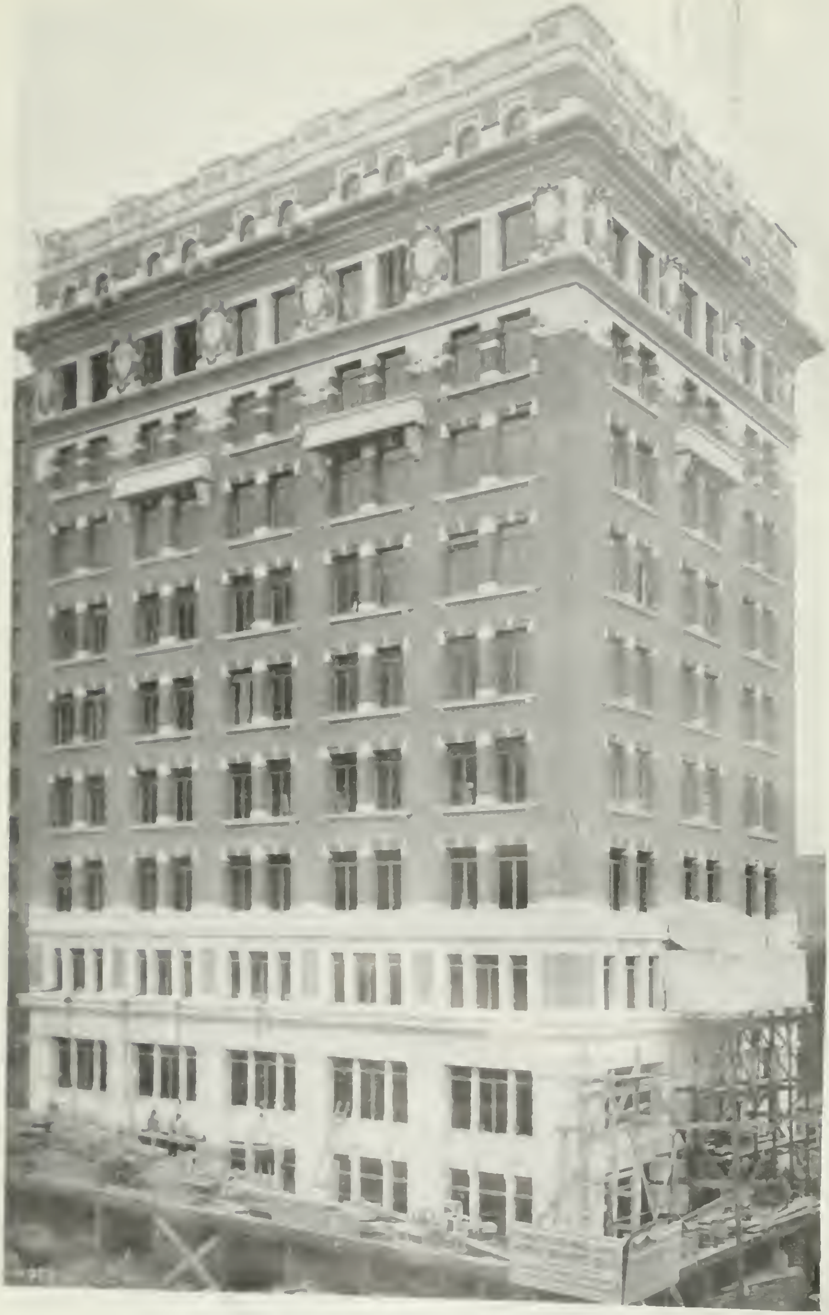

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CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 1. LONDON, JANUARY, 1912.

EDITORIAL NOTES.

A REVIEW OF THE PAST YEAR'S WORK.

THE past year, 1911, was probably not quite as important a one for the development of concrete and reinforced concrete as some of the years we have had to review since the establishment of this journal, but for all that there were certain features that call for remark, and should influence the subject beneficially.

In the first place, there was the issue of a second report by the Joint Committee on Reinforced Concrete. We have dealt with that report at length and consider it an important document, eminently useful to those concerned.

Next, the London County Council, after many delays, has finally made up its mind as to what regulations it is going to recommend the Local Government Board to accept for reinforced concrete in the county area. These regulations, as we have already remarked, are much belated, and there has been unnecessary delay, due to a certain extent to the divergent opinions of the professions concerned, but the regulations are at last in print, and this is a matter of congratulation that has to be put to the credit of the year 1911.

The fact that the Concrete Institute organised a summer meeting may also be considered a step in the right direction. This summer meeting, it is true, was not as successful as it deserved to be as far as attendances were concerned, but there were certain external influences, the then impending Coronation and the hot weather, that militated against indoor meetings.

The successful Building Trades Exhibition, with its interesting exhibits of concrete and reinforced concrete, must count as one of the important events of the year, and these exhibitions continue to be one of the most instructive of their class.

Lastly, we would record that the year 1911 actually presented us with an instance in which the Local Government Board, regardless of its inane policy of conservatism, has actually granted a 30 years' loan period on a piece of structural work on which reinforced concrete was used. A concession of this kind on the part of that particular Board is certainly epoch-making, and though this belated acceptance of one particular application does not in any way allow us to congratulate ourselves that Whitehall has at last awakened to the fact that reinforced concrete is at least as durable as timber, it at all events gives future applicants a precedent to work upon.

As far as new work in Great Britain in the materials to which this Journal devotes itself is concerned, concrete has well held its own, but reinforced concrete has somewhat marked time. On the other hand, work in hollow concrete blocks has been making rapid strides. In other parts of the world all departments of concrete and reinforced concrete have been making great progress, and this has been particularly the case in the United States, in Germany, and in Austria.

As to the impending year, 1912, the outlook appears to be a satisfactory one as far as work, scientific research and legislative measures are concerned, and, speaking generally, the building world in the coming year on the professional, official and industrial side may expect to be busier than in the year just concluded.

REINFORCED CONCRETE IN THE STORAGE OF EXPLOSIVES.

An important Report has recently been published by Major Cooper-Key, H.M. Chief Inspector of Explosives, in which the use of reinforced concrete in buildings intended for the storage and manufacture of explosives is dealt with at some length.

The Report in question, No. CD5,702, which is obtainable from the agents of H.M. Stationery Office, relates to the explosion of a store belonging to the Granville Colliery Co., at Swadlincote, and should be of the utmost importance to the many mine owners who have to store explosives on a considerable scale.

We give the clauses of the Report that deal with the question in full, but, generally speaking, we think we should preface it by saying that our impression is very much to the effect that reinforced concrete lends itself particularly to the class of work here under review, with one proviso, however, that the reinforcement of the concrete should be of as slight a character as practicable, so that in case of explosion the walls of the building would be pulverised as much as possible.

Apart altogether from the structural view, in the event of explosion, it should also be borne in mind that a reinforced concrete building is generally a safer one for the prevention of unlawful entry, as the weak feature in the usual building used for storage of explosives is the roof, which if of the ordinary construction of slate or tile can so easily be entered by those who wish to obtain access to the explosives without the knowledge of others.

In giving the extracts of the Report, which should be of such importance to those concerned and interested in reinforced concrete, we think we should point out that the Explosives Department of the Home Office has long had the reputation of being able to give sound technical advice, and the present holder of the office of Chief Inspector, by his reports on this and other kindred subjects, has clearly shown that the tradition of his Department is not only being well maintained, but usefully developed.

The following is the extract in question :—

EXTRACT FROM THE REPORT.

> I now come to the question of the best material for the construction of a magazine or store. The principles we advocate in respect of the construction of "danger buildings" is fairly well known—viz., that in *working* buildings, where the danger arises inside, the lightest possible material is the best, but that in *storage* buildings, where danger is more likely to arise from the outside, a substantial construction is preferable. Thus no fault can be found with the method adopted in respect of this store, which indeed was built under the eye of the local inspector and approved in the strongest terms by the late Colonel Cundill a year later, but the result of the explosion shows only too plainly that although the

stout construction may be, and probably is, an excellent preventive of accidents, yet if an accident does occur this same stout construction adds considerably to its effects. The store did not contain more than half the quantity of explosive that might lawfully be kept in it, yet heavy masses of brickwork were hurled far beyond the distance (100 yards) which a store of this division is required to maintain from public highways and dwelling houses with consent of occupier, and in different circumstances serious casualties might have resulted. Fortunately, explosions of magazines or stores are very few and far between, and I do not suggest that any such drastic remedy is required as to compel licensees to pull down existing buildings and re-erect them of some material other than brick, but I think it might be well before building a new store to consider whether it may not be possible to devise some method of construction which, while sufficiently substantial to afford protection against unauthorised entry, would not in the event of an explosion lead to so searching a bombardment of the neighbourhood as occurred in the present instance.

In a lecture delivered by the late Mr. Oscar Guttmann at a meeting of the Society of Chemical Industry on June 1st, 1908, it was stated that experiments had been carried out at Cummersdorf, Germany, with a view to ascertain the effect of an explosion in a magazine constructed of concrete with an aggregate of fine gravel. Two such magazines were built and it was found that the explosion of one and a half tons of gelatine dynamite in one of these buildings failed to project any of the building material to a greater distance than 50 metres, and Mr. Guttmann suggests that if Expanded Metal or stout wire netting were inserted in the concrete not only would the above result be attained but a building so constructed would also be lightning proof provided the metal in the walls were thoroughly well earthed. Without unreservedly accepting these views, inasmuch as the shattering effect of a large quantity of dynamite may produce a very different result to that which might follow the explosion of a propulsive explosive such as gunpowder, I cannot but think that there is, even on theoretical grounds, a strong probability that a material such as concrete or cement which is not cut up into small blocks by lines of least resistance, as is the case with brickwork, would in all circumstances give more satisfactory results. It must not be forgotten that a lawfully situated store is surrounded by a zone free from "protected works," and it is better, therefore, from the point of view of the safety of the public, that large blocks should be projected to distances within that zone, and that only an occasional fragment large enough to do harm should be thrown beyond it rather than that quantities of bricks should be evenly distributed, as in the present case, to distances equal to nearly double the radius of the statutory "danger" zone. I would suggest, therefore, that where practicable and in places where the margin of unoccupied ground does not extend beyond the minimum distance required by law, stores should preferably be built of fine gravel concrete with an arched roof of the same material, the construction to be as light as possible with due regard to security—a condition which can be furthered by the use of Expanded Metal or stout wire netting in the concrete.

THE CONCRETE INSTITUTE "TRANSACTIONS."

THE Transactions of the Concrete Institute for the session 1910-11 were issued last month. The Transactions, which form a very well-printed and illustrated volume, present a useful summary of the Institute's work during the session that ended in June last, and the papers and discussions there carry the reader's mind back to the autumn of 1910, and thus refresh his memory on much that, if perhaps somewhat out of date, will nevertheless make interesting reading to many of the members of the Institute.

B 2

Why these Transactions could not have been divided up into at least two parts and issued while the subject matter was moderately new we do not know, and even an interval of five months between the last paper read in June and the date of issue appears to be more than ample.

The change of size makes the volume a convenient one to handle and the folding plates are excellent.

As to the contents, we have dealt with the subjects under review in earlier issues and would, therefore, not recapitulate any of the features beyond pointing to the fact that for careful study the *in extenso* reproduction of the different papers to be found in the Transactions is invaluable. Another useful feature are enlarged notes and extracts from technical papers.

It is pleasant to observe from the Transactions that a good start is being made with the formation of a reference library.

OBITUARY NOTICE.

THE LATE MR. ISAAC CHARLES JOHNSON.

On the 29th November last Mr. Isaac Charles Johnson passed away in his 101st year. Having regard to his great age his end was not unexpected, but the termination of his long and honourable career has created a feeling of regret and sorrow which has extended from his own immediate circle to all those who are in any form connected with the manufacture and use of Portland cement at home and abroad. Mr. Johnson was associated with the cement trade almost from its birth and was prominently identified with it throughout the whole of his life, having founded the well-known firm of I. C. Johnson & Co., Limited, of Gateshead-on-Tyne and Greenhithe-on-Thames.

From his early association with the firm of J. B. White & Bros., of Swanscombe, to the day of his death, covering, we believe, a period of some 80 years, the development of the trade has been nothing short of marvellous. In this development Mr. Johnson undoubtedly took a prominent part, and although we cannot accept the view which has in recent years been given publicity that he was the actual inventor of Portland cement, there is no doubt that he materially assisted in the perfection of the product and in the method of manufacturing it. Undoubtedly the cement which Mr. Johnson made in his early days was a great improvement upon that first manufactured by Aspdin, and so in turn is the cement of to-day infinitely superior to that produced in the middle of the last century.

Mr. Johnson was entitled to receive full credit for his early observations and for the ingenuity with which he applied them in various directions, always with the view to an improvement in the value of the product and in the means of producing it efficiently and economically. With others in England and on the Continent—notably in Germany—and more recently in the United States, he played his part, and in the early days a very prominent part, in bringing the Portland cement trade into its present prominent and invaluable position among the leading industries of the world.

On the occasion of his hundredth birthday in February last he was the recipient of a testimonial and illuminated address, which was subscribed to by all the leading manufacturers of cement in the United Kingdom, and similar testimony of the regard in which he was held was received by him from many foreign countries.

REINFORCED CONCRETE RACE-STAND, BLACKPOOL.

One of the most useful applications to which reinforced concrete can be put is in the construction of race and football stands, where great stability and strength are of the utmost importance. The particulars below were prepared for us by Mr. Albert Lakeman, Hon. Medallist, Construction.—ED.

THIS stand, which is erected at the Clifton Park Race-course, is constructed entirely of reinforced concrete, at a total cost of about £10,000. It has accommodation for about 20,000 people, and was commenced in April of the past year and finished by the middle of June, this time including the design of the stand and the preparation of all the working drawings, the contractors being under a heavy penalty in the event of non-completion in the contract time.

The total length of the stand is about 466 ft. and the depth about 50 ft. 9 in., the columns being spaced across the depth at intervals of about 12 ft. 6 in., giving three intermediate rows of supports. About 96 ft. in length is given up for the accommodation of members, with a bar and smoking-room on the ground floor, and a drawing-room at the back of the second floor. Luncheon-room accommodation is also provided on the ground floor. Another 83 ft. in length is appropriated to Tattersall's, with a large luncheon-room on the ground floor, while the remainder of the space is given up to ordinary spectators, with large luncheon-room and bars on the ground floor.

The Kahn System of reinforcement was employed, and a working load test on one bay, which was made after six weeks, did not produce any deflection. The total height of the stand is 36 ft., the first-floor level being 13 ft. 9 in. from the ground level at the back, the latter point being 6 ft. lower than the ground level at the front. The drawing-room floor level is 8 ft. 6 in. above the first-floor level, and the stand is covered, as shown in the section illustrated on page 8, with a cover which projects 23 ft. from the extreme back of the stand, or 10 ft. 6 in. from the column centre which carries the fulcrum beam. The two back rows of columns only extend to the full height of the stand, as the remainder are stopped at the underside of the stringer beams below the tiers which they carry. In the Members' and Tattersall's portions of the stand the treads are 2 ft. 9 in. wide, and the risers are $13\frac{1}{4}$ in. high, giving a total number of nineteen treads in the height. The same conditions apply in the 10s. 6d. stand, but in the 2s. 6d. stand the treads are reduced in width to give a total number of twenty-seven in all.

The front row of columns have a free length of only 4 ft., and these are 8 in. square, reinforced with four $\frac{3}{8}$-in. Kahn Rib bars, with links at 8 in. centres in the height. The second and third rows are about 9 ft. and 15 ft. high respectively, and these are increased to 10 in. square, reinforced with four $\frac{1}{2}$-in. Rib bars, with links as above. The fourth and fifth rows are the two back rows, which, as

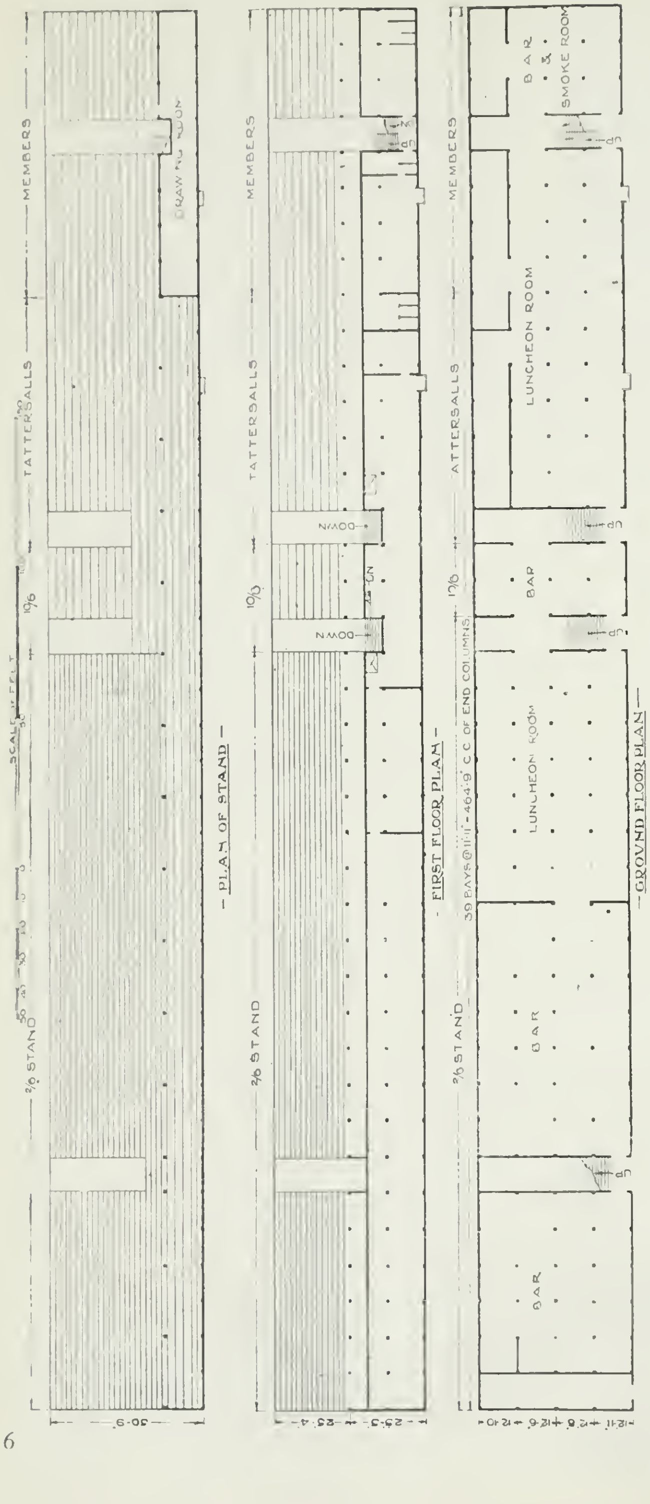


Fig. 1. Plan of the Stand.

Reinforced Concrete Race-Stand, Blackpool.

previously mentioned, extend to the full height of the stand, which is 36 ft., and these have beams connected to them at various intervals in the height. The size of these columns varies from 8 in. square, reinforced with four $\frac{1}{2}$-in. Kahn Rib bars at the top, to 12 in. square, reinforced with four $\frac{5}{8}$-in. Rib bars at the bottom.

The risers are 3 in. thick, and these act as beams, being reinforced on the lower edge with one $\frac{1}{2}$-in. Rib bar carried through straight for the whole length and one $\frac{3}{8}$-in. bar bent up at the ends and carried over stringer beams into the next bay for a distance of 18 in. to take the shear and negative bending moment. Stirrups are provided at 27-in. centres, these being $\frac{3}{4}$ in. by $\frac{1}{16}$ in., carried

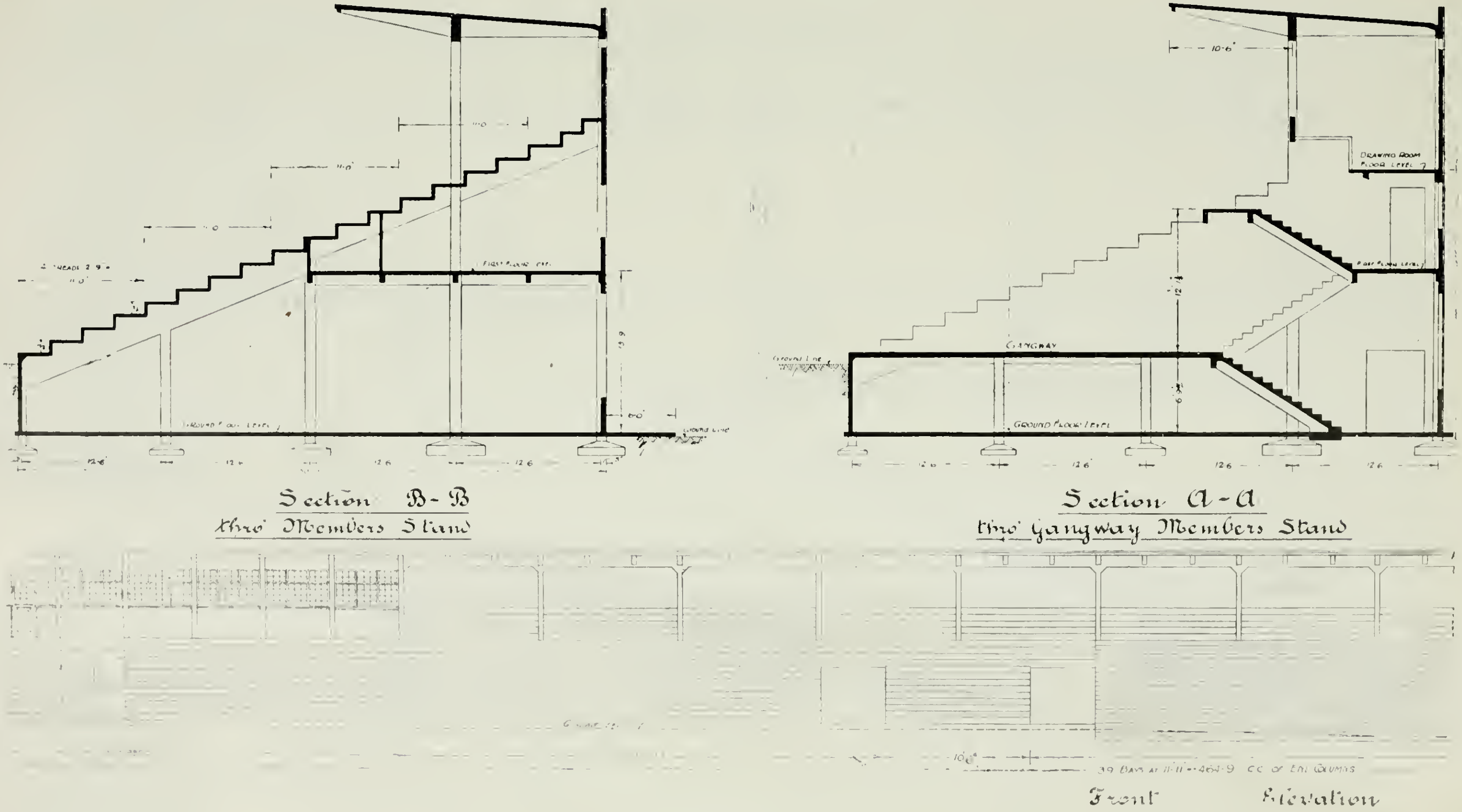


Fig. 2. Sections and Elevation.
Reinforced Concrete Race-Stand, Blackpool.

well up into the concrete. The treads which are carried by these riser beams are formed as slabs $2\frac{1}{2}$ in. thick, reinforced with $\frac{1}{4}$-in. rods at 9-in. centres in the lower surface, hooked up and down at the ends into the risers. Temperature rods $\frac{1}{4}$ in. diameter are provided in each tread, 18 ft. long, with a lap of 12 in. at the ends with the adjoining rod. The stringer beams vary in size as shown in the drawings, the largest being 24 in. deep and 8 in. wide, reinforced with two $\frac{1}{2}$-in. Kahn Rib bars and one $\frac{3}{4}$-in. Kahn trussed bar.

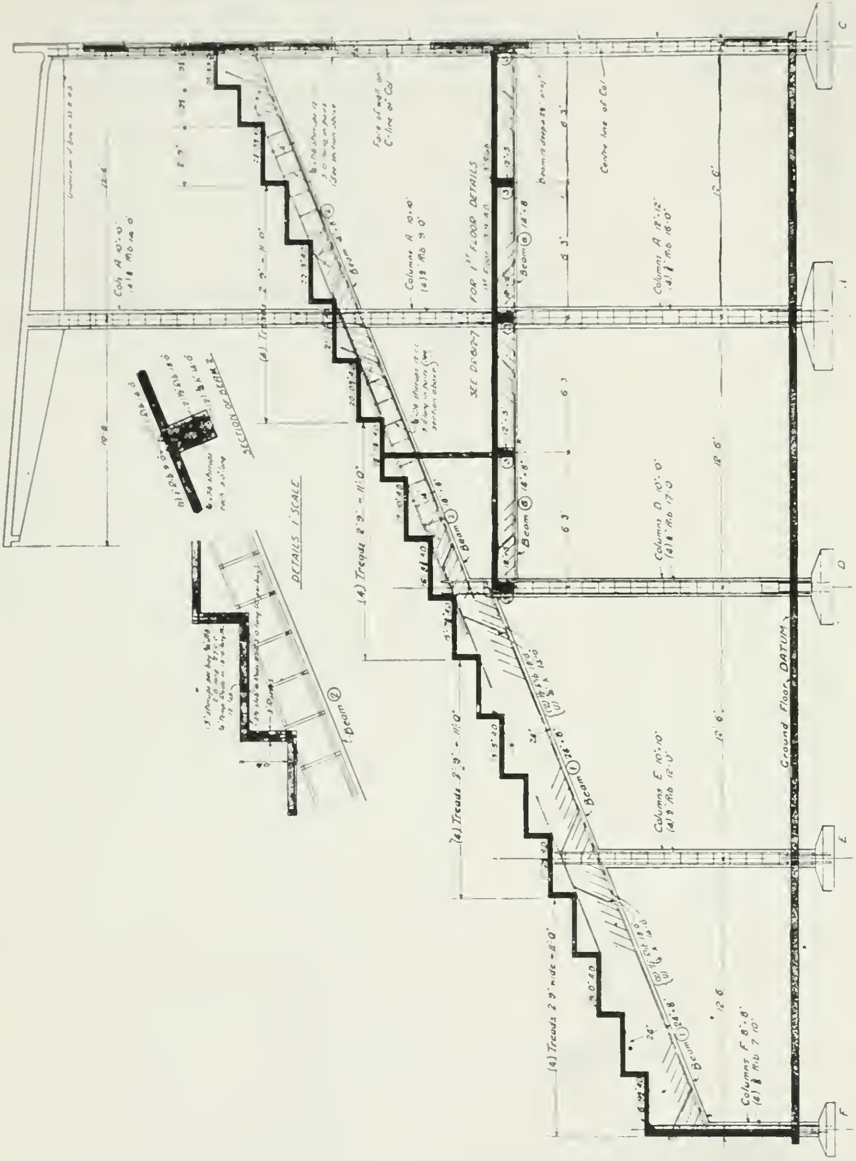


Fig. 3. Cross Section.
Reinforced Concrete Race Stand, Blackpool.

Owing to the great length of the stand, expansion joints were provided, three of these dividing the structure into four separate divisions. These joints, a detail of which is shown in the details illustrated on this page, are about ½ in. wide, and it will be seen that the beam and slab are merely supported on the haunches at the connection with the columns, these haunches being reinforced

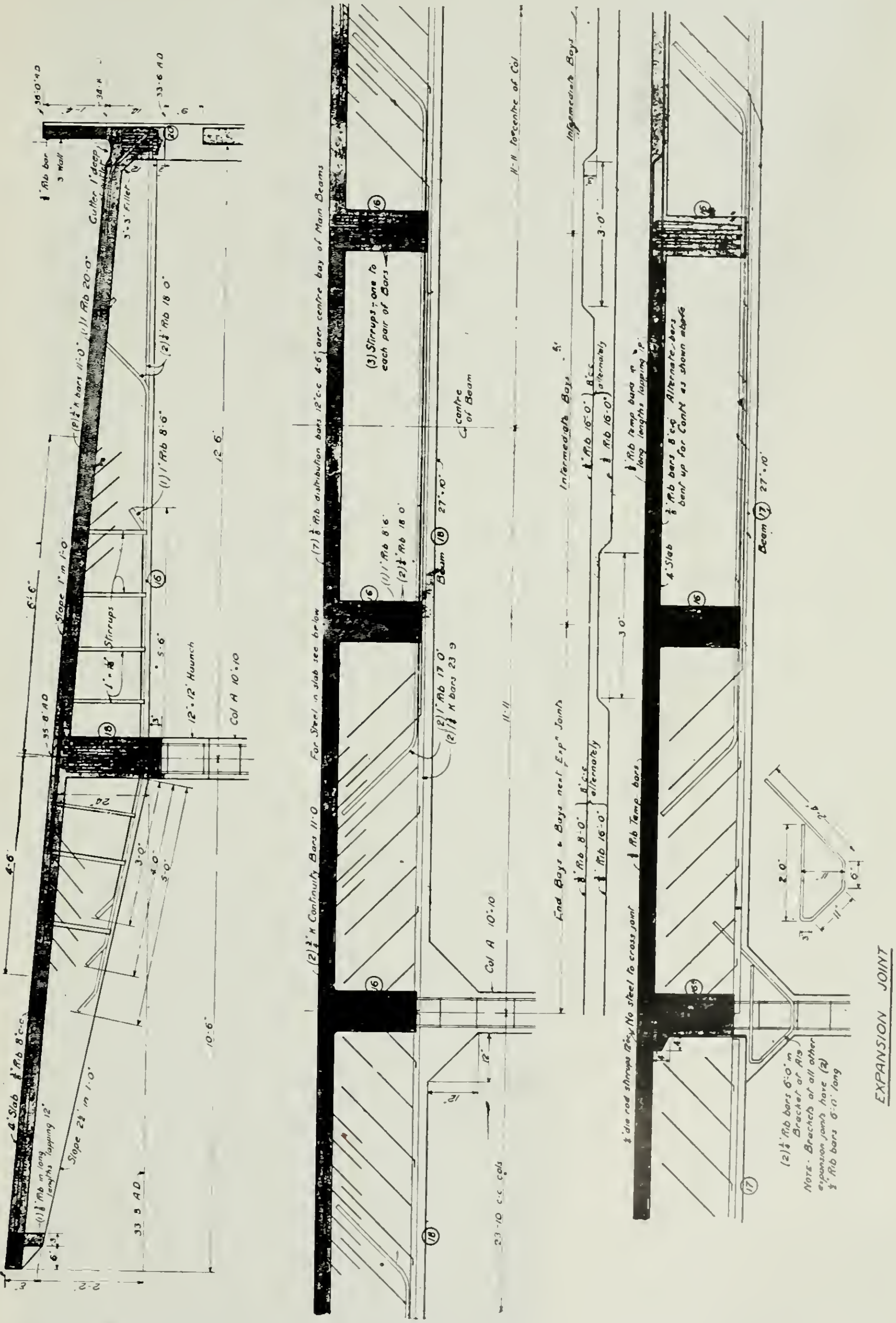


Fig. 4. Details of Beams and Cantilevers.
Reinforced Concrete Race-Stand, Blackpool.

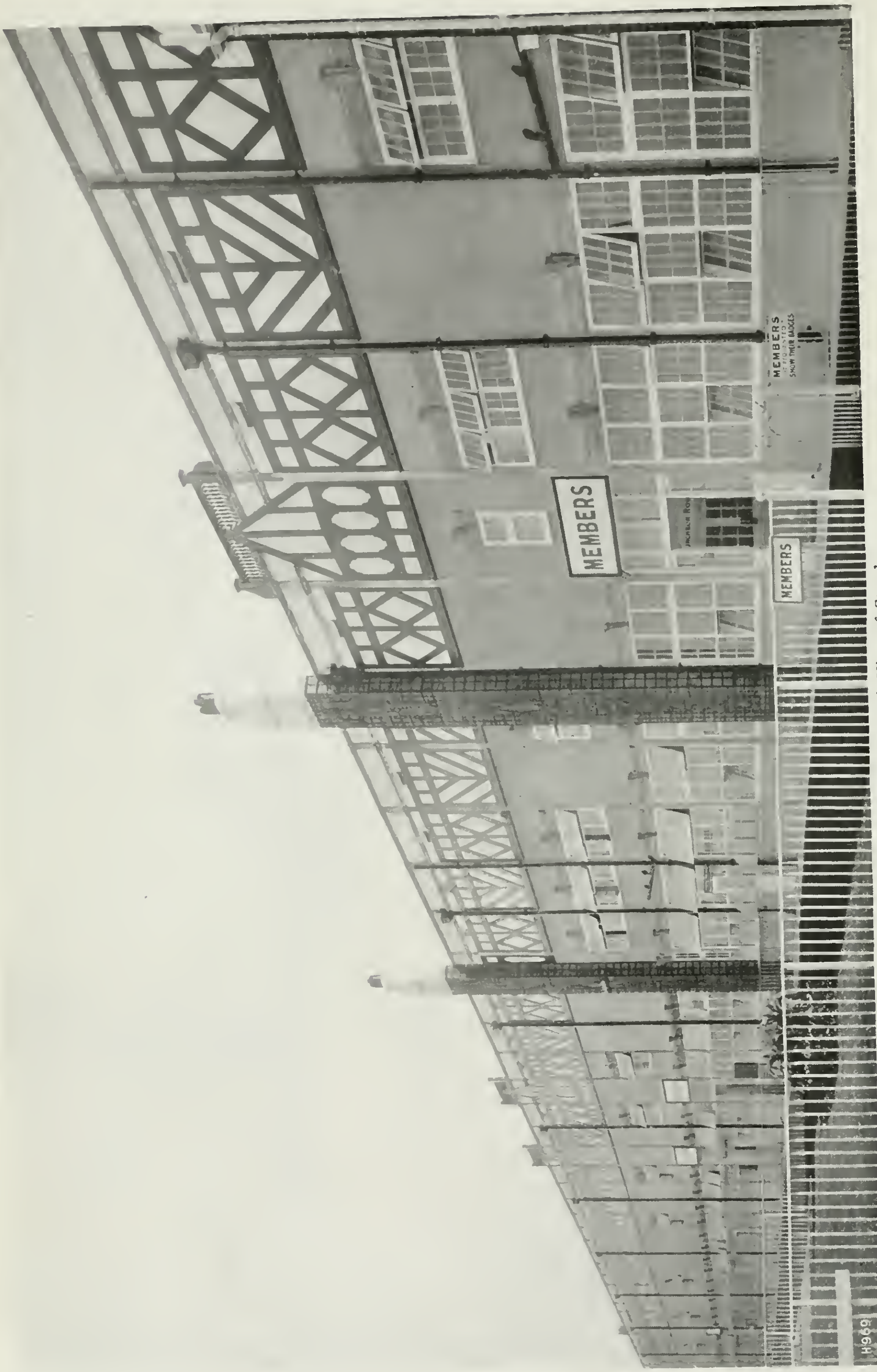


Fig. 5. Back View of Stand.
REINFORCED CONCRETE RACE-STAND, BLACKPOOL.

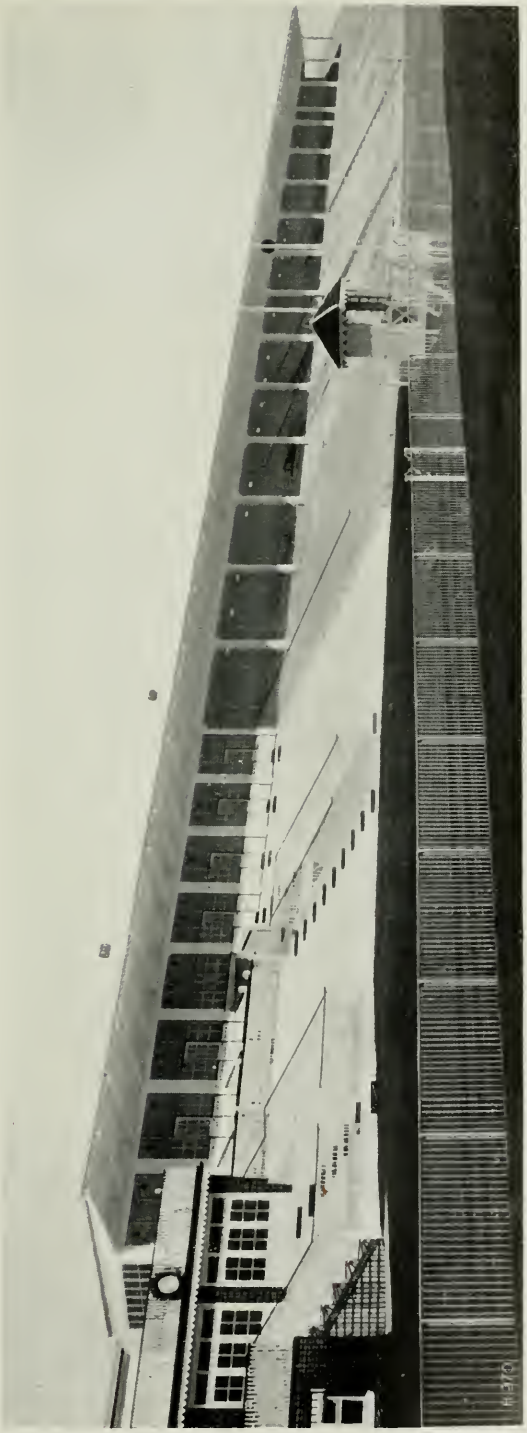


Fig. 6. Front Elevation.
Reinforced Concrete Race-Stand, Blackpool.

with two Rib bars to enable them to act as short cantilever brackets. No steel is, of course, allowed to cross the joint.

The cover at the top of the stand is designed with sufficient strength to allow same to be used by spectators if necessary, thus serving a double purpose. It is formed with a slab 4 in. thick, having a slope of 1 in. in 12 in., and finished with cement and sand in equal parts applied before the concrete was set, and no waterproofing material was required. A gutter is formed at the back next a 3 in. concrete wall which is 16 in. high and forms a finish. The projection of the cover from the fulcrum beam is 10 ft. 6 in., while the length of the tailing-down portion is 12 ft. 6 in. The slab is reinforced with $\frac{3}{8}$-in. Rib bars at 8-in. centres, the alternate bars being bent up over the fulcrum beam for continuity, and temperature bars $\frac{3}{8}$ in. diameter are provided at 2-ft. 6-in. centres. The fulcrum beams are 2 ft. 3 in. deep and 10 in. wide, reinforced with two 1-in. Rib bars and two $1\frac{3}{4}$-in. Kahn bars. These beams have a span of about 24 ft., and each one carries two cantilever beams, which are spaced at 8-ft. centres. These latter have a maximum depth of 24 in. next the fulcrum beam and a minimum depth at the extreme outer edge of 8 in., with a width of 10 in., the reinforcement consisting of one 1-in. Rib bar and two $\frac{3}{4}$-in. Rib bars in the lower surface and one 1-in. Rib bar and two $\frac{3}{4}$-in. Kahn bars in the upper surface, the latter being continuous in the cantilever and tailing-down portion. Stirrups 1 in. by $\frac{1}{8}$ in. are provided to connect the two sets of reinforcement, one stirrup being provided for each pair of bars.

The back of the stand is treated with half-timber work, which gives a relief from the flat treatment which would have otherwise been obtained, as shown in the photograph of this portion, which is illustrated on page 10.

The finished view of the front is shown on page 11.

This structure is an admirable example of design in reinforced concrete, and the use of this material for a race-stand is so superior to timber that one can hardly conceive the latter material being adopted in the future for buildings of this kind. The maintenance required is practically nil with the use of reinforced concrete, and it combines with strength the quality of being fire-resisting.

The engineer for the work was Mr. Arthur Hindle, A.M.Inst.C.E., of Blackpool, and the reinforced concrete was designed by the Trussed Concrete Steel Co., Ltd., of Caxton House, Westminster. The contractors were Messrs. J. Parkinson and Sons, Ltd., of Blackpool.

FIRE-RESISTING CONCRETE.

SOME POINTS FOR CONCRETE DESIGNERS.

By HAROLD G. HOLT, A.R.I.B.A.

The question of using concrete for fire-resisting construction is very much under discussion, and the article we present below should therefore be of interest to our readers.—ED.

THE systematic study of fire-protection in its two aspects of fire-resistance and fire-extinction has been wonderfully advanced during the past ten years, due chiefly to the efforts of the British Fire Prevention Committee in this country and to various American technical bodies.

With regard to fire-resistance in which constructional methods are systematised to afford the best results under fire, nothing has been more remarkable than the decided tendency toward the introduction of first-class concrete, of a suitable nature, for the construction and protection of all classes of building, and to this may be attributed in no small measure the greatly extended use of reinforced concrete as a sound constructional method.

The employment of mild steel in the main structural members of most modern buildings demands a satisfactory form of protection against fire. The liability of steelwork exposed to high temperatures to warp and twist and pull the very heart out of a building, causing the collapse of floor areas and the consequent providing of horizontal openings inducing draught and the spread of flames and smoke, is well known and need not be further dealt with in the present article; suffice it to say that the adequate protection of all steelwork in a building is one of the cardinal points of modern fire-resisting construction.

CONCRETE versus TERRA-COTTA.

In England the palm for such protection has been generally awarded to concrete; although we have had an invasion of the American terra-cotta tile—and some forms of the latter have occasionally proved capable of sustaining little damage under severe tests, more particularly where employed, in the case of floors, in conjunction with some form of reinforced concrete web or joist for the bearing member—concrete still holds its own, and is even more likely to do so in the near future. This is due to a better understanding of the nature of concrete required for fire-resisting work, better supervision of workmanship, superior design and increased facilities for obtaining dependable Portland cement.

In America concrete systems are the only serious rivals of terra-cotta tile construction. There the construction of commercial buildings of great height, combined with a peculiar dryness in the atmosphere and consequently the ever-present danger of the fire hazard, render the question of steel protection an urgent one.

Terra-cotta tile has been in great demand in the United States for a considerable time, one of the reasons for this being, no doubt, that it meets the demand existing there for some form of "quick-erection" construction; the hollow tiles, being manufactured previous to erection, are dry and after being applied in the building are practically ready for plastering. Ample opportunity has thus been given for observing the behaviour of this form of construction under severe fire in some of the large conflagrations that have taken place—notably, Baltimore, San Francisco, Illinois, and other large centres. Its behaviour under these circumstances has not borne out many of the statements of the interested manufacturers and often of insurance officials, who are beginning to seriously revise their opinions, which at one time were wholly in favour of tile construction.

DEFECTS IN TERRA-COTTA TILE.

Terra-cotta intended for use as a fire-resisting material is of three kinds: dense, porous, or semi-porous. It is made usually in the form of hollow blocks, the inner and outer skins being connected by webs of the material (much in the same way as the glazed ornamental terra-cotta used for facing many buildings here). The blocks are of numerous shapes, designed to meet varying applications, such as floor arches, steel beam or stanchion protection, partitions, etc., etc. They are made of refractory clay burned in a kiln at a temperature of 2,000° to 2,500° F.; the clay used in manufacture is mixed with sawdust, and the latter is burnt away during the firing process, thus producing the required degree of porosity.

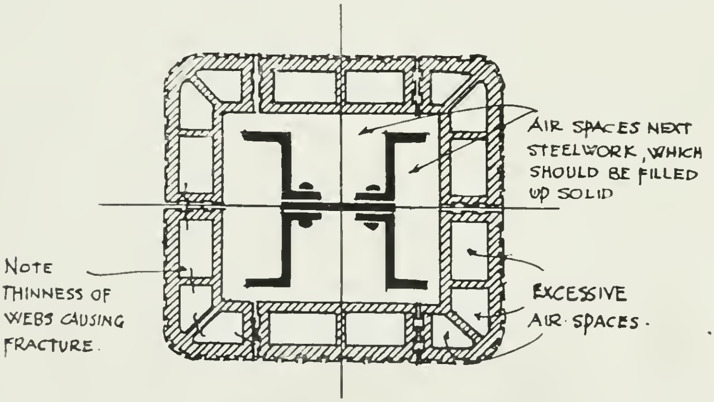

AIR SPACES NEXT STEELWORK, WHICH SHOULD BE FILLED UP SOLID
NOTE THINNESS OF WEBS CAUSING FRACTURE.
EXCESSIVE AIR SPACES.

Fig. 1. Terra-Cotta Tile Casing to Steel Columns.

The comparative failure of many terra-cotta systems in actual fires in America has been clearly proved and may be explained.

As might be expected *dense* terra-cotta tile is the worst offender, a somewhat porous material being, in general, a better fire-resistant than one of a non-porous character. Dense terra-cotta failed so badly in the great San

Francisco fire that the Commission appointed by the Society of American Engineers to investigate the construction of the buildings reported that they considered " dense terra-cotta " to be " the least valuable form of fire protection." On account of its greater structural strength, as compared with porous types, this kind of terra-cotta has been used to a very great extent in the construction of large flooring areas, particularly in arched forms and in the vital construction of many buildings; hence the great number of collapses and failures due to fire of which we have heard in connection with many so-called " fireproof " buildings. It is an axiom of fire-resisting construction that the main columns or piers into which steel enters *must* be adequately protected from fire, otherwise the protection of steelwork in floor beams and flooring areas becomes a waste of material and money. Terra-cotta has been used alone for the protection of steel columns or stanchions in various American buildings, and has failed to perform its function in numerous instances, chiefly for three reasons : *firstly,* the unequal expansion of terra-cotta with steel, causing dislocation of the blocks when high temperatures are reached and the consequent opening of joints and exposure of steel—also the crushing of arch blocks where confined between walls and steel beams, etc. ; *secondly,* the use of the wrong type of terra-cotta, viz., dense, instead of semi-porous or porous ; *thirdly,* thinness of the material and excessive air spaces in the hollow blocks. Notably, in the Baltimore fire the large air spaces in the column protection blocks acted as sufficient non-conductors of heat to destroy the homogeneity of the inner and outer skins by reason of the extreme difference of temperature which obtained in the outer and inner faces and the consequent setting up of stresses sufficient to crack off the outer face, leaving the inner shell an easy medium for conveying the heat to the steelwork and producing the opposite effect to that intended. This will be clearly seen from *Fig.* 1.

The terra-cotta manufacturers have to some extent now remedied these defects and introduced webs of greater thickness, increased the thickness of the shells and reduced the area of air spaces ; this has been done at slightly increased cost of manufacture and extra weight. Some forms of porous or semi-porous tile may be used in buildings with advantage at times. A test by the British Fire Prevention Committee as long ago as 1905 showed that a partition erected of *porous* 2½-in. slabs, plastered ⅜ in., would withstand a fire of two and a half hours, with temperature rising to 1,980° F., and then prevent the passage of a water-jet from a steam fire-engine. The area of partition exposed in this case was 80 ft. super. A floor test in the same year on a floor put forward by the National Fireproofing Company, of Pittsburg and London, constructed of *semi-porous* terra-cotta with wire truss reinforcement in the joints of the blocks, gave good results. The fire did not pass through the floor, which deflected ⅜ in. at the maximum during the test and regained its normal condition after the test ; classification as full protection for four hours was gained. In the above two tests, which are favourable to terra-cotta tile, it should be noted that the latter were either porous or semi-porous, and not the *dense, hard quality* which is chiefly used in American buildings, as previously mentioned ; the floor tested would be greatly benefited by the wire truss reinforcement which is bedded in Portland cement mortar. It was, moreover, of the flat type, not arched, and erected by the foremost manufacturers of tile in America and in a most expensive and somewhat complicated manner. Arched floors, which are very common in this type of construction, are open to the objection of side-thrust exerted on steel girders and walls; attempts to counteract this are made by inserting steel tie-rods from girder to girder in the floor bays, and failures have been caused by the insufficient protection of such rods in these floors.

The chief objection, however, to the application of any form of terra-cotta to steelwork lies in the unequal expansion of the two materials under high temperatures, which, given time enough, will cause sufficient damage to wreck a building, particularly where great fluctuations of the fierceness of the fire occur, such as would happen in a high wind.

ADVANTAGES OF CONCRETE AS PROTECTION FOR STEEL.

The great points in favour of concrete as a fire-resistant applied to steel of any form (whether in the nature of rolled steel joists, girders or columns, or in combination with rods and reinforcements as a reinforced-concrete system) is that it expands almost equally with steel, its thermal conductivity is not great, and, owing to its natural bond with steel, it does not readily admit of the expansion of the latter.

The co-efficient of expansion of steel is, roughly, ·0000066 per deg. F. Various experiments have been conducted to show the co-efficient of expansion of concrete; Professor Hallock, of Columbia University, found that for a 1 : 3 : 5 concrete the co-efficient was ·0000065 per deg. F., and experiments at Perdue University demonstrated that for 1 : 2 : 4 broken stone concrete the co-efficient of expansion varies from ·0000052 to ·0000057 per deg. F. The figures are probably known to the majority of my readers; they have, of course, a very practical bearing on reinforced concrete design generally, but they are of the utmost importance in considering the fire-resisting value of concrete when applied to steelwork, and they have been emphasised in many of the fire-test results that have been obtained.

NECESSITY FOR SUITABLE CONCRETE AS A FIRE RESISTANT.

The use of "suitable" concrete for fire-resisting purposes has already been alluded to, and it is necessary to point out that all kinds of concrete are not well adapted for protective work.

Various factors enter into this, particularly that of the type of aggregate to be used (affecting seriously the thermal conductivity of the concrete), and also the nature of the cement, proportions and mixing and general workmanship.

Not only does the thermal conductivity of the concrete depend largely on the aggregate employed, but the *amount of protection* to be afforded steelwork depends on the type of concrete employed, and this is a point frequently overlooked by concrete designers.

Important experiments to determine the thermal conductivity of various types of concrete have been conducted in America, chiefly by Prof. Woolson. He employed concrete tubes two months old, one face being exposed to heat and having thermo couples embedded at varying distances from this face; the temperature was carefully regulated and gradually raised to 1,500° F. in less than one hour, and kept thus for a period of four hours. The results I do not propose to give in full detail, but they may be briefly indicated to show the main conclusions arrived at.

The following table shows the rise in temperature of various cubes under test at stated distances from the heated face:

Type of Aggregate in cube.	Time heating.				Distance of Thermo Couples from heated face.
	60 min.	100 min.	180 min.	300 min.	
Trap rock	225°	300°	925°	1325°	1 in.
	200°	300°	675°	800°	1·5 in.
Cinder...	150°	225°	350°	550°	1·5 in.
	200°	450°	850°	925°	3 in.
Limestone	250°	500°	1050°	1250°	3 in.
Gravel	750°	1100°	1350°	1600°	2 in.
	525°	975°	1325°	1500°	3 in.

It will be seen that gravel concrete is a dangerous material from a fire-resisting standpoint; it allows of the penetration of heat and rise of temperature after a short time to a degree at which the concrete will commence to lose strength rapidly. Not only so, but any steel reinforcement near the surface would also become dangerously heated, and sufficiently so to cause failure. The "cinder" concrete shows up well; this type approximates to our coke-breeze and clinker concretes, though it is probably not used in America with the care that it demands and usually receives at the hands of English contractors, and is open to the suspicion of containing unburnt coal and other deleterious and dangerous substances. In general, the experiments showed that steel members in a building should receive a protection of *two inches* of good-class fire-resisting concrete, though where gravel types of aggregate are used this should be increased to four inches as a minimum in exposed situations, such as main girder soffits, columns with rectangular protection, etc. The extreme importance of these results was not at once grasped, but it was gradually realised that concrete would prove a very economical and reliable fire-resisting medium; it meant reduced first cost, saving of space in column protection, and consequent possible reduction in the size of bays in factory and office building. Further tests have been conducted which show the low heat transmission of Portland cement concretes and mortars; thirty panels of building materials were erected at the Structural Materials Laboratories of the United States Geological Survey, St. Louis, and of the various stones, bricks, terra-cotta tiles, and concretes, the latter showed the lowest thermal conductivity and cracked the least of all the materials. The tests showed also, however, the desirability of introducing reinforcement in concrete walls to distribute the effects of expansion. Tests showing the comparative average compressive strength of concrete composed of 1 cement, 2 sand, and 4 of $\frac{3}{4}$-in. broken stone,

(1) trap rock, (2) limestone, have been made by Prof. Woolson, with the following results:

AVERAGE COMPRESSIVE STRENGTH (IN LBS. PER SQ. IN.).

Aggregate.	Normal	500° F.	750° F.	1000° F.	1250° F.	1500° F.	1750° F.	2000° F.
Trap rock	1900	1850	1800	1400	1200	1000	1000	700
Limestone	1800	1250	1200	1000	900	800	750	Crumbled on cooling
Time when temperature reached ...	—	Min. 55	H. M. 1 35	H. M. 2 15	H. M. 2 55	H. M. 2 55	H. M. 3 35	H. M. 3 35

The trap rock specimens showed comparatively little deterioration in strength up to 750° F., and then gradual decrease to 1,750° F. On the other hand, limestone showed a considerable decrease in strength at 500° F., and beyond 750° F. this loss is more marked than in the case of trap rock specimens.

Experiments conducted to show the modulus of elasticity of concrete after heating demonstrated that this became very low, but in all cases the value increased with increase of load.

This is probably due to the closing up of many fine cracks by the application of the load, as on cooling the rapid contraction caused many fine cracks to develop, and the compression observed would not therefore be a true deformation of the concrete at the various pressures.

Tests on seven floor panels have been conducted by the British Fire Prevention Committee to show the value of various concrete aggregates from a fire-resisting point of view and to record the effect of a fire of three hours' duration, the temperature to reach 1,800° F., but not to exceed 2,200° F., followed by the application of water for two minutes. For full particulars I must refer readers to Report No. 101 of the Committee. The aggregates under test were furnace slag, broken brick, broken granite, burnt ballast, coke breeze, furnace clinker, and Thames ballast. The latter came out worst of all; the slab was cracked in many places, the top curved down 2 in., and the underside of the slab curved downward 1¼ in., badly cracked and much washed off by water. The results of this test showed the poor fire-resistance of Thames ballast aggregate concrete and thus confirmed a previous test conducted on August 17th, 1905, at the Committee's testing station, when a floor constructed of 5 in. thick Thames ballast concrete (between 4-in. by 1¾-in. rolled steel joists at 2-ft. 4-in. centres, and supported by 10¼-in. by 10¼-in. broad flanged beams at 7-ft. centres, finished on the top with boarded floor and wood joists) failed badly.

The floor was constructed and put forward by Messrs. H. T. Skelton & Co., London, and the time allowed for construction and drying was 11½ weeks. The test occupied four hours, the maximum temperature reached being 720° F.; the floor was loaded at 280 lb. per sq. ft., the maximum deflection during test being 7 3-10 in. and permanent set after test 4⅞ in.

In 22 minutes after commencement of the test the soffit of the concrete began to split off in patches. In 75 minutes and 100 minutes respectively the concrete to the soffit of the beams fell; towards the end of the test the concrete in one bay began to fall in, and the wood floor was ignited; fire and water (which was applied for 5 minutes) passed through the floor, which was badly damaged. The floor failed at the end of 3⅔ hours.

In the panel tests above referred to the best results were obtained by the *coke-breeze* aggregate concrete, which was not curved down or cracked, but about 1 in. was washed off the underside, in parts, by water. *Burnt ballast* concrete panel neither cracked visibly nor curved, but about 3 in. was washed off the underside, in parts, by water. *Furnace slag* and broken brick aggregate concretes follow, with slight cracks and ¼-in. deflection in both cases. The *furnace-clinker* slab was cracked across in two places and curved downwards ⅜ in. on top; the underside was not curved downwards, one slight crack visible, and the concrete was pitted in places 1 in. deep. The *broken-granite* slab was cracked in three places on the top, underside curved down ¼ in., with slight cracks. In the above test (which was carried out privately by the British Fire Prevention Committee) the area of slab tested was 31½ sq. ft.; the test being on seven panels 2 ft. 7 in. wide, the concrete, 5½ in. thick, being supported by 6-in. by 4½-in. steel joists, encased in coke-breeze concrete in each case, 10-ft. span. The time allowed for construction and drying was seven weeks, the load per sq. ft. 224 lb.; the bays were plastered on the underside, and plaster commenced to fall off the beams 10 minutes after test began, and this continued to the end. All slabs remained in position.

It is interesting to note that a test on a *broad-flange beam floor* by Messrs. Skelton (exactly similar in construction to that with Thames ballast as the concrete aggregate which failed so badly—with this difference, that the concrete aggregate was *furnace clinker*) successfully passed it as "full protection" after 4 hours, neither fire nor water passing through the floor, the maximum deflection during test being only 7-20 in., and the permanent set *nil*. In the test small pieces of concrete came off the soffit, and the surface of the beams was eroded by the water application, but the maximum temperature reached was 1,890° F., exceeding the standard temperature by 90° F., and being a difference of 170° F. above that at which the Thames ballast floor failed.

It should be explained that the British Fire Prevention Committee set up a standard of requirements for floors which must be passed by those under test before admission to their respective classifications. This is as follows:

STANDARD CLASSIFICATION FOR FLOORS.

PROTECTION.		Least duration of Test.	Minimum Temp. of Fire to be reached during Test.	Minimum Load per Super. Foot Distributed.	Minimum Superficial Area under Test.	Min. Time for application of Water under Pressure
Temporary	Class A ...	45 min.	1500° F.	Optional.	100 sq. ft.	2 min.
	Class B ...	60 ,,	1500° F.	,,	200 ,, ,,	,,
Partial ...	Class A ...	90 ,,	1800° F.	112 lbs.	100 ,, ,,	,,
	Class B ...	120 ,,	1800° F.	168 ,,	200 ,, ,,	,,
Full ...	Class A ...	150 ,,	1800° F.	224 ,,	100 ,, ,,	,,
	Class B ...	240 ,,	1800° F.	280 ,,	200 ,, ,,	,,

Among other floor tests one or two of special interest to reinforced concrete designers will be worthy of note, briefly and chiefly regarding aggregates.

A floor by Messrs. E. Coignet, Paris and London, of *clinker concrete,* 6 in. thick, with rod reinforcement, obtained the A classification; the maximum deflection recorded was 5 6-10 in. and permanent set 5 7-16 in., also concrete to beam soffits was knocked off by water application, exposing the rods; neither fire nor smoke passed through the floor. A somewhat similar floor of *blast-furnace slag* and Portland cement concrete, but with the beams at 5-ft. 9-in. centres in place of 7-ft. 5-in. centres in the former test, obtained the B classification and gave better results; the deflection at maximum during test was 4 3-10 in. and permanent set 1¾ in. In this case, again, *the concrete on beam soffits was knocked off on water application and the steel rods exposed;* this was due to the two main 1½-in. and 1⅝-in. rods in the beams in both cases being too close to the exterior face of the concrete, leaving only 1¼ in. of concrete protection on the connecting ¼-in. rods in the most exposed situation. Another floor, by the Patent Indented Steel Bar Co., London, obtained the B classification. This was another *blast-furnace slag,* sand and Portland cement floor, with indented steel-bar reinforcement; the maximum deflection during test was 9-10 in., permanent set after test ¾ in.; neither fire nor water passed through the floor, but *again the concrete to the beam soffits was knocked off* where struck by the water-jet *exposing the steel bars,* and the floor soffit eroded where struck by the jet. The New Expanded Metal Co. put forward a floor which gained B classification. This was constructed with 10-in. by 5-in. rolled steel joists at 7-ft. 8-in. centres, and of concrete composed of *broken* firebrick and broken brick and cement reinforced with expanded metal. At the end of one hour a portion of the concrete casing to one beam fell, but no metal was exposed and no damage was done to the soffit of concrete by fire or water; maximum deflection during test 1¾ in., permanent set ⅞ in.

A test on a different type of floor, one of the reinforced concrete hollow block or tube floors that have sprung into prominence of late, shows that these are to be quite relied upon when precautions as to the casing of steel reinforcements and the use of aggregates is considered. The floor in question was formerly known as the Herbst (Berlin) Floor, and is known now in this country as the Armoured Tubular Floor. Full classification B was attained in the test; the floor was constructed with webs of shingle concrete with steel reinforcement, and filled in with coke-breeze concrete tubes and covered on top with 1½ in. of concrete, soffit plastered. The maximum deflection during test was ½ in., permanent set after test 7/16 in. The water application produced no effect beyond eroding some of the plaster on the soffit. Of ordinary steel and concrete floors the following two tests are interesting. The first test was on a floor of coke-breeze concrete, 5 in. thick, filled in and supported by 5-in. by 4½-in. rolled steel joists, 2 ft. 6 in. apart, centre to centre, the top and bottom flanges of the joists being exposed and flush with the concrete faces. After 20 minutes the steel joists began to deflect until a maximum deflection of 10½ in. was recorded. After 1 hour and 20 minutes the bulk of the concrete in the two outer bays collapsed, the concrete in the other two bays being slightly disintegrated on the underside. The floor failed at the end of 80 minutes.

The other floor was of coke-breeze concrete filled in between 7-in. by 3½-in. steel joists, the concrete being 9 in. thick and covering the lower flange soffits with 2-in. protection. After 12 minutes plastering fell off, but beyond this no other effect was observed (no observation was taken of deflection during test); there was no permanent set, but the soffit of the concrete was disintegrated about $\frac{3}{4}$ in. where struck by the water. The fire did not pass through the floor; the *maximum temperature reached was 2,230° F.* There was no load on this floor.

Many further tests have been undertaken by the British Fire Prevention Committee to observe the action of fire on concretes of various aggregates, but it would occupy too much space to give any further detailed particulars of tests. In general, the conclusions arrived at point to the necessity for about 2 in. of protection for steel reinforcements, and to the general excellence of coke-breeze concrete *per se* as a fire-resistant. I have personally observed that coke-breeze concrete floors have stood very well after fires in cotton mills in Lancashire (where such floors have been in use for the last fifteen years in conjunction with small steel joists and main girders), wherever proper protection has been afforded to the steel by about $1\frac{1}{4}$ in. of concrete and $\frac{3}{4}$ in. of plaster on floor soffits, and 2-in. to $2\frac{1}{2}$-in. protection of main steel beam flange soffits. It is a general practice in the north to include a small percentage of clean, broken, hard-burnt brick in the aggregate, which counteracts the over-porosity of the breeze. Such concrete is, of course, not suitable in the majority of cases for application in reinforced concrete construction proper. Pan-breeze and smith's ashes are universally condemned for concrete, and great care must be observed in the use of breeze that a very small percentage only of sulphur be present. Thoroughly hard, well-burnt furnace clinker and blast-furnace slag, free from lime and sulphur, have come out well in tests, and may be recommended for fire-resisting concrete, as can aggregates of clean, broken brick, broken porous terra-cotta, and pumice or lavas. The latter must be free from decomposed parts and from all soft and organic matter; they should be dense, thoroughly vitrified, not scoriaceous, and free from marked cellular structure. Burnt ballast of clay when sufficiently burnt approximates to brick, and may be employed for fire-resisting concrete, but is not to be recommended in general, because it is usually insufficiently burnt and often contains free lime, sulphur and other deleterious matter. With regard to other natural material for aggregates, gravel and ballasts are unsuitable; broken limestones and granites must be broken to small pieces, otherwise they are apt to be dangerous as aggregates in concrete subjected to high temperatures.

Further points that should be noted in connection with concrete for fire-resistance are that (1) in all the tests and results observed a rich mixture proved better than a poor one; (2) a wet mixture proved better than a drier one; (3) aggregates broken in small pieces proved better than those which would not pass a $\frac{3}{4}$-in. ring. With regard to the first, it is, of course, well known that fine-ground Portland cement is an excellent fire-resistant, and provided that there is sufficient sand and aggregate to prevent cracking, the richer mixtures are better; also, of course, giving increased strength, allowing extra margin for careless workmanship. In some of the tests referred to when the concrete blocks cracked or spalled under heat previous to water application, the damage was found to be greater in the dry mixtures containing the greatest percentage of sand, and it was also noted that the richer mixtures warmed up more slowly than the others. In tests on some concrete hollow blocks it was also observed that one cause of weakness in the blocks was a too dry and lean mixture; this, coupled with the thinness of webs, proved them insufficiently strong to withstand the stresses due to rapid expansion on the heated faces. (It should be observed, *en passant,* that hollow concrete blocks should be manufactured with *sufficiently thick webs* to withstand these stresses, and if of *richer concrete* they can be made to pass practically any tests required.)

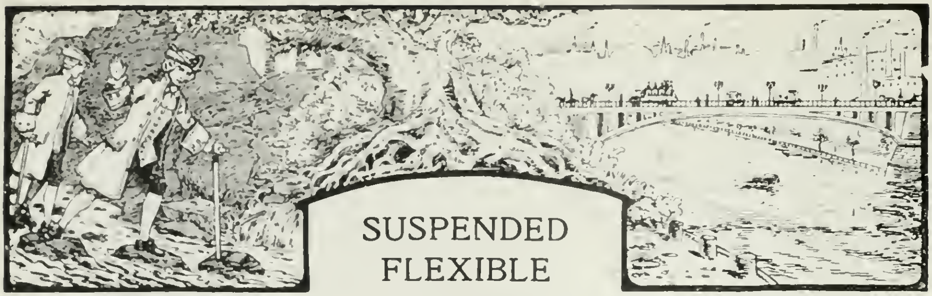
With regard to point 2, it has been discovered that before concrete will disintegrate under fire-exposure the large amount of moisture chemically combined in the setting of the cement (about 20 per cent. to 25 per cent. of its weight) has to be driven off by the heat, and the vapour thus driven off must be evaporated thoroughly from the concrete pores before the latter become sufficiently hot to crumble. This evaporation is a very slow process, and it is due to this fact that concrete will resist extremely high temperatures for some hours and provide all the fire-protection necessary in any building before a fire dies out; it would become disintegrated ultimately at much lower temperatures were they continued for any length of time. The damage which has occurred to floor panels and blocks under severe fire tests has been probably caused at temperatures lower than those ultimately reached, in many cases (and previous to water application) owing to this water of crystallisation being driven off from the face, leaving a thin coat of dehydrated mortar which would probably form a coating sufficient to present further dehydration deeper in the concrete. A suggestion is contained in the last report of the Committee on Treatment of Concrete Surfaces, of the National Association of Cement Users (U.S.A.), regarding the possibility of treating concrete, to render it more fire-resisting, with a solution of 1 part 40 degrees Baumé of sodium silicate, 3 parts water, allowing to dry for 24 hours, and then a further immersion in the same solution and a further drying for the same period. It is stated that concrete so treated will stand a very much higher temperature *without dehydration of the cement* than concrete not so treated. It is evident that a field for investigation opens here, for if such an enamelling treatment can be practically and commercially achieved, and it can be demonstrated after complete and thorough tests to greatly increase the fire-resistance of concrete and decrease its already low thermal conductivity, the popularity of concrete as a fire-resistant will be still further increased, and possibly insurance ratings on concrete buildings would be further decreased.

With regard to point 3, it is, of course, a necessity to employ concrete with aggregates broken finely for filling into reinforcements in reinforced work, and, as already pointed out, it is especially necessary in the case of natural broken stones used as aggregates in fire-resisting work. Limestones and granites are particularly liable to failure in fires. The former are calcined and converted into quicklime, losing all cohesion when exposed to fire and water; the latter cracks and flies to pieces or crumbles to a fine sand when exposed to high temperatures. Sandstones are superior to either for standing against fire, but become disintegrated after exposure. Granite, of course, is particularly in demand in a great many reinforced concrete works, and otherwise limestone forms an excellent aggregate.

It would be better to employ them for the structural duties in reinforced concrete work and then to afford them an extra protection against fire of a mixture of concrete having, say, coke breeze and broken brick as the aggregate. The reason that aggregates broken in smallish pieces will stand fire and water application better than concrete with aggregates of larger size may be explained as follows:—Rocks are poor conductors of heat, and whilst the outer shell of a stone may be quickly raised to a high temperature if exposed to fire the inside may be fairly cool; if the stone be suddenly cooled, say, by water application, contraction of the outer shell taking place will cause pieces to be thrown off owing to the great internal stresses set up. Where small stones are used the chances of extreme differences in temperature are not so great.

PROTECT STEEL ADEQUATELY.

In conclusion, I wish to particularly emphasise the especial necessity for protecting all steel members in reinforced concrete work. There is a consensus of opinion amongst architects and engineers that the minimum protection of suitable concrete that should be afforded steel in main beams is 2 in., and in main floor areas of 1 in. for rod reinforcements, as the very minimum. This is somewhat vague and does not make allowance for varying types of aggregate and is to be taken as an average rule only; in the case of floor areas of reinforced concrete of broken stone I think that in addition to the ½ in. or so usually allowed on the soffit below the rods an additional protection of 1 in. of fire-resisting concrete should be allowed, or a protection of 2 in. of stone aggregate concrete in all as a minimum. The amount of special protection, if any, or the total amount of covering to rods, etc., should, I consider, be very clearly stated in all specifications and tenders for reinforced concrete work.



SUSPENDED FLEXIBLE CENTERING.

And a description of its use in the construction of the Chickahominy River Bridge, Richmond, Va., U.S.A.

By PHILIP AYLETT, A.M.Am.Soc.C.E.

The following particulars of a new form of centering should be carefully studied by our readers, as any new method claiming to reduce expense and to give efficiency in reinforced concrete work is one that deserves particular attention.—ED.

Of all classes of centering the most difficult and costly to erect, maintain and remove, and that affording the greatest obstruction to traffic, both by land or water, is undoubtedly the ordinary rigid timber centering, usual in the construction of arch bridges of masonry or concrete. The centering of bridge arches is constantly liable to damage by fire and water, resulting in the injury or complete destruction of the incompleted superimposed structure.

Even small movements in centering from timber shrinkage or settlement prior to or at the time of striking the centering have proved prolific sources of permanent injury or even complete destruction. In several known cases the centering was destroyed by fire, the origin of which was probably due to shavings, chips, etc., left by carpenters constructing the timber falsework.

In a number of cases on record, washout of centering by a sudden freshet has resulted not only in disaster to the structure being built, but also other structures spanning the same stream below have been swept away by the floating falsework timbers. Loss of life and property has frequently resulted from the proximity of centering (supporting overhead structures) to railway tracks under traffic.

In almost every case of the loss or damage to arch bridges during construction, or subsequent development of cracks, the failure, weakness, or even small settlement incidental to the construction, maintenance or removal of the so-called rigid timber centering is usually responsible.

In addition to the risks, delays, etc., involved by the use of rigid centering, its expense is usually the largest factor in the cost of arch bridge construction; its cost, as well as its risks and dangers, increasing out of all proportion with increase of span and depth. The usual demand in the engineer's specification for "absolutely rigid and unyielding" centering in arch bridge construction has become both difficult and dangerous to meet under the heavy loads of modern long-span arches, and this demand must, of course, add largely to cost by requiring expensive foundations, skilled construction, low working stresses, and, consequently, heavier timbers and more of them in order to eliminate shrinkage settlements and insure the rigidity and strength required during the long periods of falsework maintenance prior to striking centres.

With the most efficient management many years would elapse before the supply of heavy timbers suitable for rigid centering will increase in quantity or decrease in cost. It appears therefore that even the present generation must resort to the other alternative—viz., the use of structural steel or a combination of timber and steel. As a result of the cost of timber centering and the greater risks incurred with increase of weight, height, and span length, a number of recent concrete structures have been built with a combination centering, or with one exclusively of structural steel. Thus far, however, the use of structural steel centering for concrete arch bridges has been restricted almost exclusively to those locations where the character of the stream or crossing rendered timber and pile falsework impossible or expensive.

The first recorded use of centering entirely of structural steel for concrete arch bridge construction on a large scale was in 1909, upon the construction of five 150-ft. channel spans of the D.L. and W. bridge over the Delaware River. The Delaware River is subject to heavy floods with ice and drift. A concrete structure could, therefore, never have been considered at all but for the elimination of the ordinary pile-supported rigid centering and the introduction of structural steel falsework. This latter consisted of three centered steel arches supported upon concrete projections on the piers designed for the purpose.

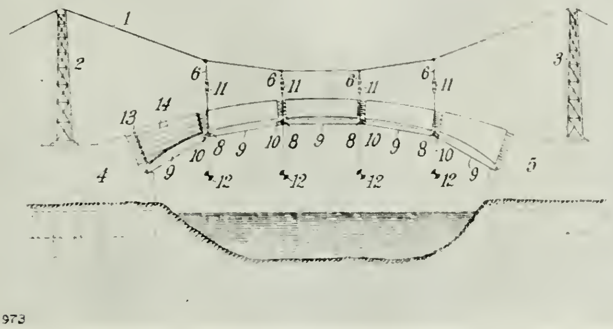


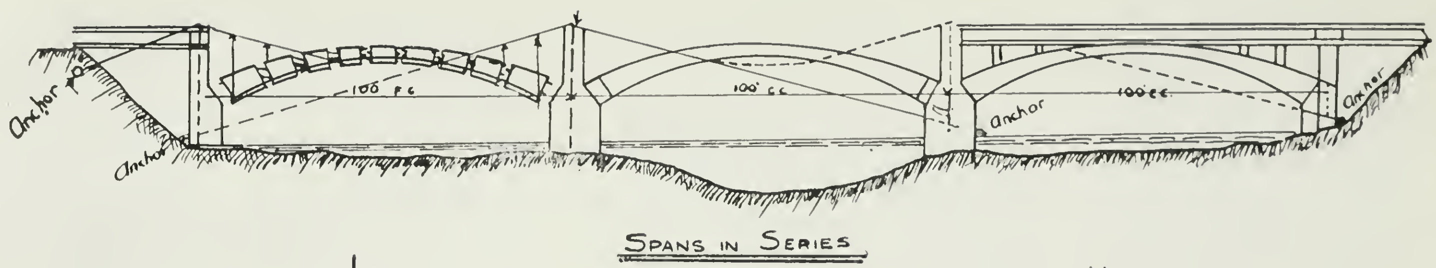
Fig. 1. Sketch showing suspended method of arch construction.
Suspended Flexible Centering.

The use of a new method of arch construction as used at the Chickahominy River, Richmond, Va., and in other work, has demonstrated that it possesses many advantages and possibilities, in addition to economy and rapidity of construction. By this method (as used at Chickahominy River), since it involves the use of no falsework whatever, the great expense, risks, etc., incidental thereto are eliminated, and thus a means is afforded of constructing concrete arch bridges over those locations heretofore considered impracticable, expensive, or altogether impossible, by reason of falsework considerations, difficulty of access, or remoteness from civilisation.

Prior to the detailed description of methods, etc., used at Chickahominy River, a brief review of some of the most important experiments, test arches, etc., preceding and leading up to its use there, will be of interest.

The method was first suggested by an actual experience with rigid falsework supported on piles during construction of a 70-ft. concrete rib span. Skin friction of the piles being reduced by scour after a heavy freshet caused the sinkage of the piles supporting the falsework, which, of course, threatened the destruction of the concrete ribs. Fortunately, the concrete ribs had been deposited in sections or voussoirs, and the spaces between them had not been filled with concrete when the settlement of fully 12 in. occurred, decreasing towards the abutments.

Jacks were placed upon the caps of the pile falsework bents which were located



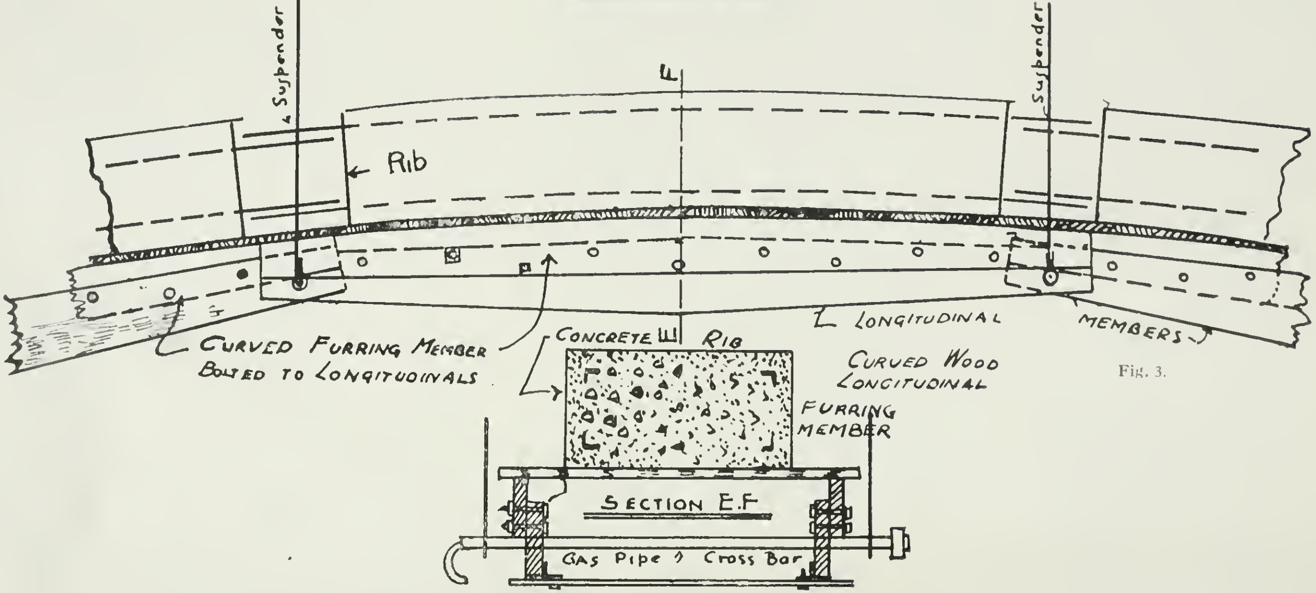


Fig. 3.

Fig. 2. Method of constructing spans in series.

Fig. 3. Elevation and section of a panel of suspended centering constructed of wood

directly under the open spaces between the concrete rib sections. The rib sections were thus raised to the true curvature, as determined by computed ordinates between the intrados and a horizontal line. The open spaces between the rib sections were then filled with concrete. Although sinkage still continued, the true curvature was maintained by the jacks until the concrete was set up and the arch became self-supporting. This structure has now carried heavy traffic for eight years.

After the success of the above emergency work, two experimental arch ribs were constructed upon falsework. One of these ribs was composed of sections which were moulded *in place,* which, after being properly adjusted for curvature, were keyed together by moulding concrete keys in the key space between the sections. The other rib was composed of *two previously-moulded* sections, supported upon hinges located at the abutments and crown. Adjustments being made prior to keying or hinging by raising or lowering the crown ends of the two sections by means of jacks, placed upon one point of support, located under the crown. When the concrete keys were set, or the pins or hinges placed between the sections as the case may be, the temporary supporting structure is removed, leaving the arch self-supporting.

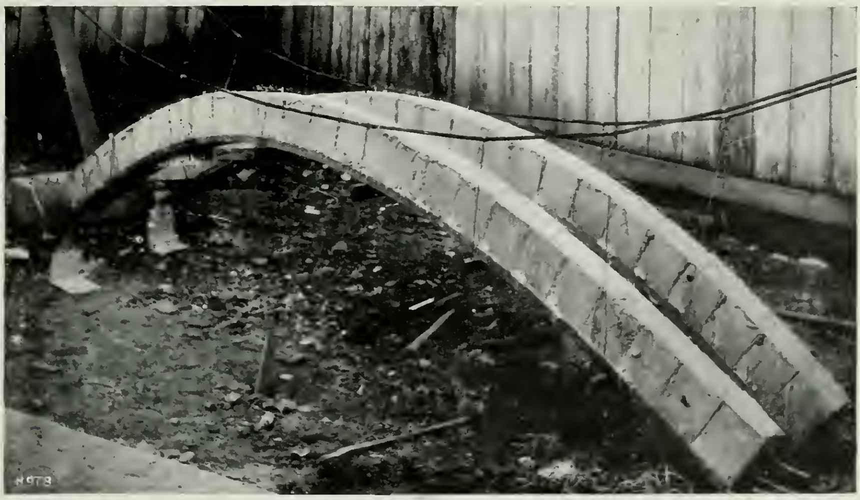


Fig. 4. Ribs supported entirely by cables, key spaces at abutments still open.
View of 10 ft. Span Test Arch.

The above method makes it possible for the temporary supporting structure to shrink, stretch, settle, or otherwise become distorted without injury to the incompleted arch, since the arch section can, at any time before keying them together, be accurately adjusted. The arch sections may be formed on the centering or moulded elsewhere and then placed in position, as stated, the sections being of any desired length.

When the sections are previously moulded, the amount of temporary falsework is much reduced, this reduction depending upon the length of sections.

The above methods, while eliminating injury due to shrinkage, settlement, etc., and reducing the amount of falsework, effect much saving and are useful in many locations.

FLEXIBLE SUPPORT OR SUSPENDED METHOD OF ARCH CONSTRUCTION.

The foregoing work and experiments suggested the Flexible Support or Suspended Method, by which arch construction has been made possible without the use of any falsework whatever. While the method can be used for almost any location, it is especially adapted to rapid and economical construction for such locations as deep gorges, rapid streams, canals, railway tracks, etc., over which, heretofore, any method of concrete arch construction has been found impracticable or altogether impossible.

Fig. 1 is a sketch in side elevation, showing the application of this method of the construction of an arch rib.

Cables indicated at 1 (*Fig.* 1) are hung from suitable supports, such as A frames or towers, located on or near the abutments. The cables are anchored by any convenient and suitable method, an ordinary deadman at each end of the cables for spans under 150 ft. being generally sufficient. Depending from the cables are the hangers (6) carrying the cross members of wood, iron rods, steel shapes or pipes. The longitudinal members (9) are supported on the cross members. They are composed either of wood or steel shapes, such as angles or channels, as desired, and rest upon the cross members, so that by lengthening or shortening the hangers (6) the series of supports can be made to assume any desired curvature, according to the design of the arch which is to be erected. Each cross bar or pivot (8) passes through holes or slots in the ends of the longitudinal members (or rests in notches cut in their bottom edges), so as to permit the curvature of the arch to be varied. The end sections adjacent to abutment skewbacks may also be suspended by hangers from the cables, but it is sometimes preferable to mount them upon suitable hinged bearings directly upon the abutments, without suspension at this point from the cables. The series of longitudinal members, flexibly jointed together, constitute in effect a flexible centering, upon which may be laid previously moulded voussoir sections of the concrete arch rib or moulds, in which the concrete rib sections can be cast directly upon the suspended centering.

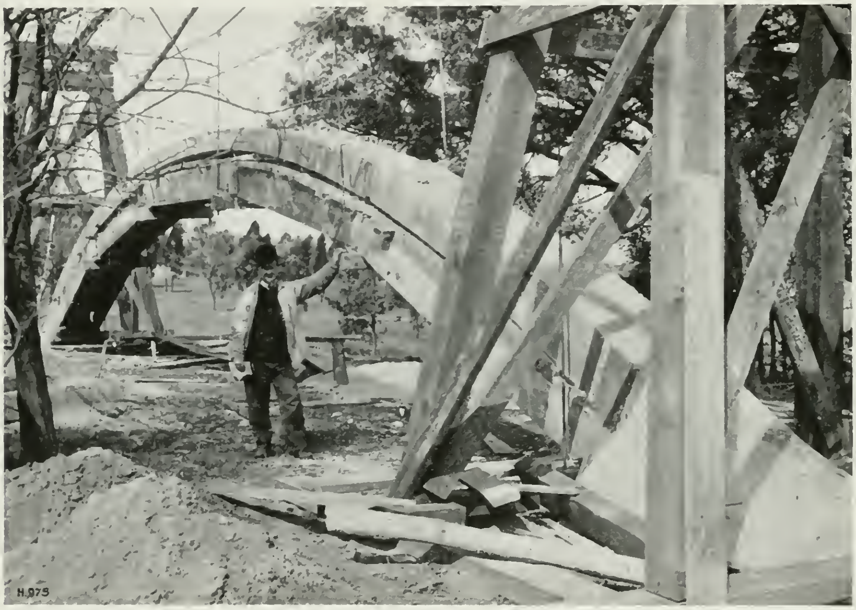


Fig. 5. Suspended flexible centering struck.
VIEW OF PORTSMOUTH 53 FT. SPAN TEST ARCH.

The length of the hangers is determined (approximately) in advance, so that when the cables and flexible centering are in place the latter will have the approximate curvature, but to permit of final accurate adjustment at any time the hangers are provided with turnbuckles (11), whereby they may be readily let out or taken up as desired, so as to correct the curvature of the centering whenever necessary. It has been found satisfactory to use wire rope suspenders instead of rods, and for very heavy spans differential pulleys may be used for adjustments, without the use of turnbuckles. Hanging from the ends of the cross bars or pivots is a row of targets (12) on wires or rods of such length (determined by calculation) that when the curvature of the flexible centering has been adjusted correctly the targets will all be at the same level. Thus under the direction of a leveller on the bank of the stream, or an adjacent pier, the sag of the cables and the lengths of the hangers can be varied until the targets are all at the same level, whereupon it is known that the centering, and therefore the concrete segments (or voussoirs) of the permanent structure, are in proper form. Extending through the moulds are the reinforcing bars (14) which may either run continuously, forming lap joints in all key spaces, or be rigidly connected.

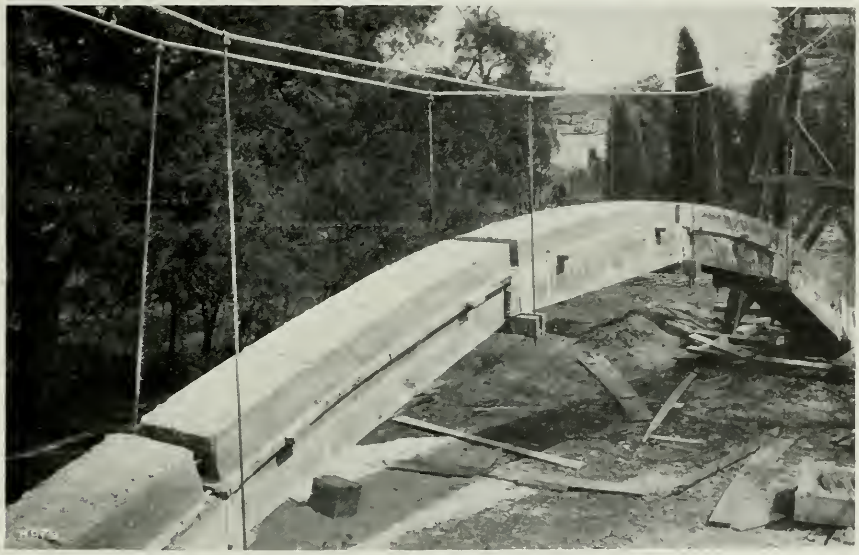


Fig. 6. Cables supporting concrete sections, key spaces not yet filled in.
View of Portsmouth 53 ft. Span Test Arch.

Symmetrical Loading Not Essential with Flexible Support.—A few years ago it was considered essential in concrete arch construction to have the arch ring a solid monolith from spring to spring. In short spans to effect this continuous work night and day was specified. With somewhat longer spans continuous work was found inconvenient and expensive, and solid barrel arches were divided into longitudinal sections of such widths that the volume of concrete contained could be deposited in a working day. Thus the completed arch is composed of monolith longitudinal sections.

With the longer and heavier spans of the last few years monolith arch ribs and barrels (even in longitudinal sections) were found impossible. Experience in arch design and construction has shown such practice an unnecessary precaution and expense, with hingeless arches. In fact, monolith arch ribs and rings are not only unnecessary, but have been found impossible of construction in modern long spans, for the reasons that proper sequence of loading rigid falsework is essential in order to guard against deformation of centering, and the impracticability of continuous work, and even disaster.

In order to secure symmetrical loading of falsework and other advantages, the most recent practice in constructing arch ribs is to divide same into alternate blocks or voussoirs. With flexible supports such division is also used to afford a means of temporary flexibility, the points of flexure (or hinges) being located between the voussoirs and at the skewbacks. By this means construction oscillations, deflections, etc., of the supporting cables have no effect upon the individual concrete rib sections or voussoirs during construction. Symmetrical loading has been found entirely unnecessary. In one case, with cable support, an arch rib was begun at one abutment and completed continuously to the other abutment. In another case the arch rib was commenced at the crown and continued to the abutments. When the concrete in the sections sets up the forms are removed and the spaces between the concrete rib sections enclosed by forms. The key spaces are then filled with concrete, embedding the reinforcement and keying the arch sections together.

Where two or more arch ribs are constructed with flexible support side by side they may be connected by transverse beams cast in forms extending betweeen the arch forms at suitable points, preferably opposite, and in communication with the key spaces, so that the transverse beams and keys will be integral or monolithic. Reinforcing bars may be provided, extending through the transverse beam forms and connected with the arch rib reinforcing bars. Since the longitudinal suspended centering members are preferably straight, furring members (preferably of wood), having their upper edges curved to the intrados of the proposed structure, are rigidly attached by bolts or otherwise to the longitudinal centering members. The furring members are also supported at their ends upon the cross members. Thus by substituting differently curved furring members and adjusting the length of the hangers, 6 (*Fig.* 1), as mentioned, an arch of any *curvature and length* may be constructed with the *same apparatus* within the capacity of the cables, etc. The arch ribs being completed, the cables and centering may be taken down and a superstructure of any desired character constructed upon the completed arch ribs, or the main cables may now be slacked off until the suspended centering drops from 3 to 5 ft. below the intrados of the arch, and in this position used as a suspended scaffold for the workmen to finish up the surfaces of the completed ribs. The cables (when not required for immediate use elsewhere after completion of arch ribs) may also be freed from hangers and suspended centering, then hoisted above the grade of the proposed roadway, and in this position used as an ordinary cableway for delivery of concrete from the mixers on shore to the spandrel and roadway forms, from *either* direction, from *both* banks of the stream, or from barges in the stream below. In the construction of heavy spans requiring supporting cables of large diameters it is best to use a special cable of small diameter for the delivery of material, this supported upon the main cable supports, thus avoiding wear and tear upon the main cables. Thus the expense and delays incidental to the transportation of concrete on runways by carts, cars, etc., is eliminated.

While the suspended system is especially adapted to the rapid and economical construction of rib-bridges of the hingeless, two-hinged, or three-hinged type, it may also be used in the construction of solid ring arches with or without hinges. By the introduction of hinges not only are the stresses more readily determined, but it also becomes possible to lighten the rib design and reduce by so much the load on the cables.

Further than this, much of the weight of the two concrete rib segments adjacent to the two abutments being supported by the abutment hinges, the weight upon the cables being thus reduced, the dimensions and weight of cables, anchorages, etc., are also correspondingly reduced. The end segments of the arch rib being pivoted by the hinges or pivots on the abutments are free to respond to the small construction oscillations of the cables due to depositing concrete, etc., in the intermediate segments and the open joints or key spaces between same.

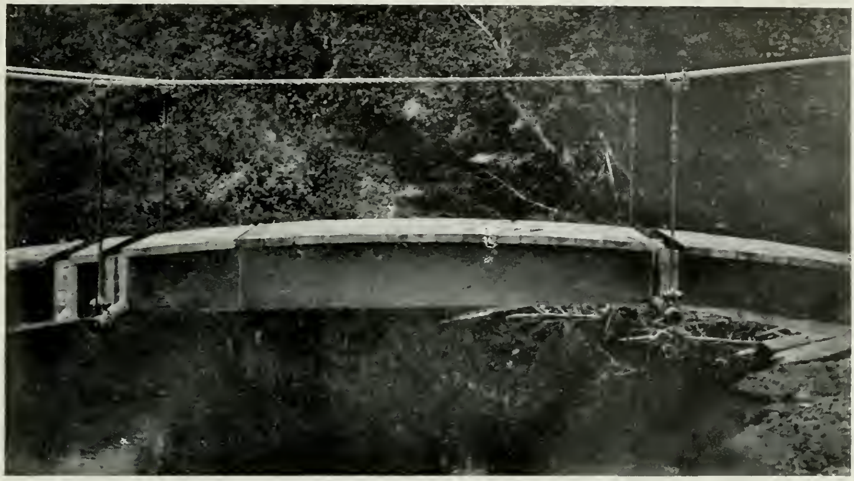


Fig. 7. View of suspended flexible centering at crown.

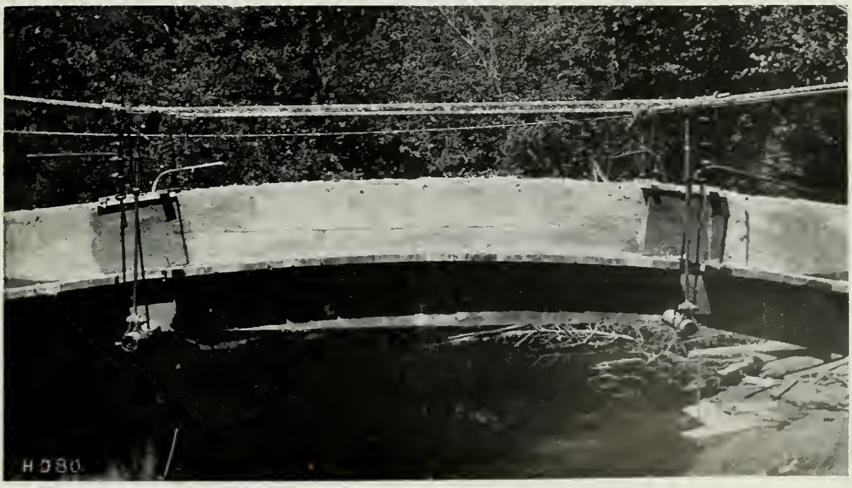


Fig. 8. View of crown after keying sections together.
CHICKAHOMINY RIVER BRIDGE.

Fig. 2 shows the application of the method to constructing spans in series, the piers and abutments of the permanent structure being used both as anchorage and cable supports. In order to pass the cables through the piers or abutments to anchorage a small pipe or box, with proper slope, is embedded in same when constructed.

Fig. 3.—Elevation and section of a panel of suspended centering constructed of wood. Curved furring member shown. To adapt to a span of different radius it is only necessary to change curvature of furring member, increasing or decreasing panels as length may require.

The reinforcing bars are here shown embedded in the concrete segments, their ends projecting into the open key spaces and overlapping the bars from adjacent segments.

Prior to actual construction of arches in concrete the method of arch rib construction by casting the voussoirs in place upon suspended flexible centering was demonstrated by constructing a number of plaster of Paris arches, all of varying curvature, and with span lengths ranging from 2 to 5 ft. It may be interesting to mention in passing that several experimental arch ribs of wet sand were moulded in these model suspended forms. These sand ribs were uninjured and self-supporting when the suspended centering was "struck," and the curvature perfect.

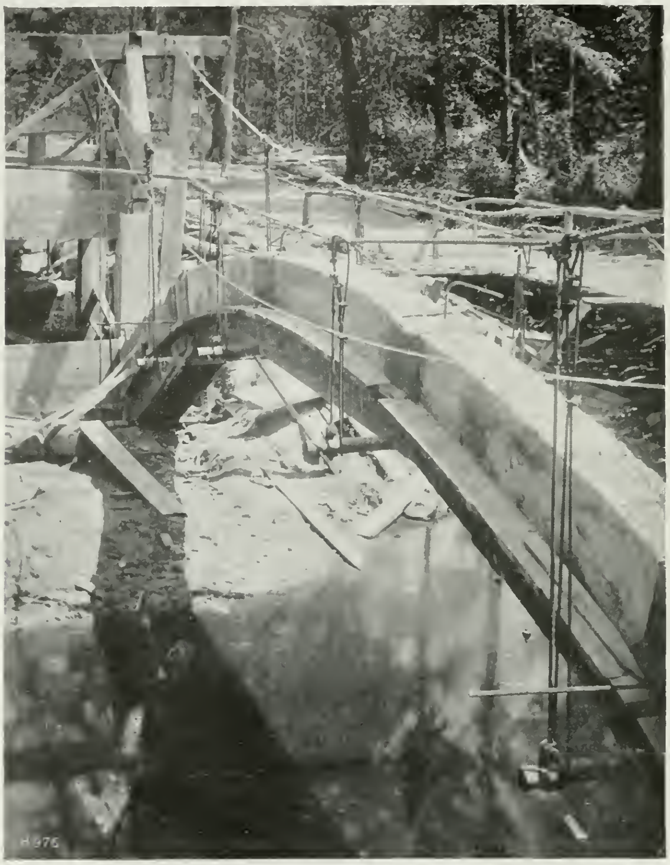


Fig. 9. Key spaces filled with concrete.
CHICKAHOMINY RIVER BRIDGE.

After the success of these experiments a two-rib concrete arch was designed and built. This had a span length of 10 ft. and 2 ft. rise, composed of two parallel ribs

12 in. apart, connected by transverse beams. The section of each rib was uniform throughout and was 3 in. in width and 4 in. deep.

The two abutments were built first. Two $\frac{3}{8}$-in. cables were suspended over the location of the proposed span and anchored to the tops of a backyard fence, after bracing same. The suspenders were made of small copper wire, tied to the wooden cross-bars and cables at proper intervals. The volume of concrete in these ribs was so small that each rib was completed prior to the initial set of the concrete.

It was therefore possible to adjust the suspenders for curvature while the concrete was in the wet state, consequently no key spaces were necessary and none provided, except those adjacent to each abutment (*Fig.* 4).

The adjustments being made the ribs were allowed to set up, after which the key spaces at the abutments were filled with concrete. The arch having reached the self-supporting stage, the centering was struck, merely by slackening the cables.

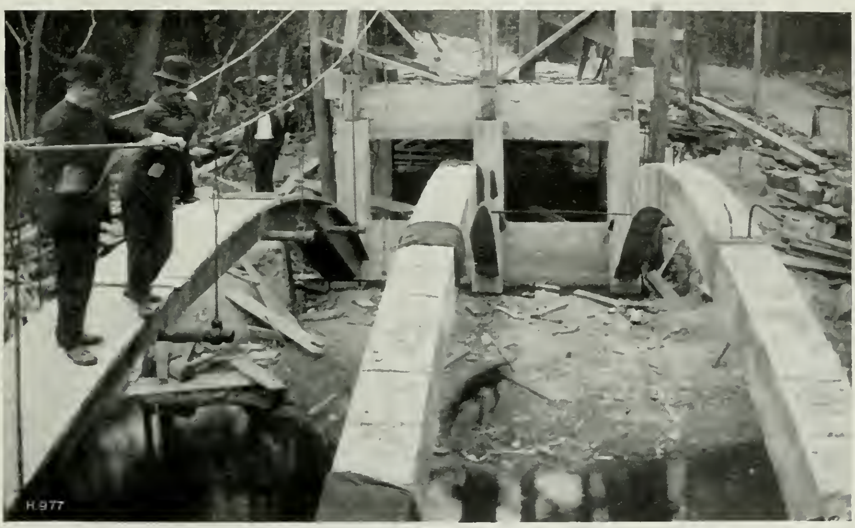


Fig. 10. Two ribs completed. Flexible centering ready for third rib.
CHICKAHOMINY RIVER BRIDGE.

It was now desired to make further tests upon a still larger scale.

With this in view the designs for an arch rib having a clear span of 53 ft. and a rise of 7 ft. were prepared, and the weights of arch, cable, sag, etc., determined.

It was found that steel cables 1 in. in diameter and suspender rods $\frac{5}{8}$ in. in diameter would support the 53 ft. concrete rib and centering, with a proper safety factor. This arch was built at Portsmouth, Va.

The rib was divided into six sections, which required six pairs of suspenders and cross-bars for their support. The abutments and towers were first built and the cables placed in position and anchored to "dead men" buried 4 ft. in the ground.

The longitudinal members of the centering were made of 2 in. × 10 in. plank, resting loosely upon the cross-bars with their ends overlapping.

To prevent slipping longitudinally a $\frac{3}{4}$-in. iron rod passed through the overlapping ends of the longitudinal members. No turnbuckles were used on the suspender rods in this case, but their lower ends were threaded 6 in., so that the proper adjustments for curvature could be made by nuts under the cross-bars. Cast-iron plumb-bobs were hung from each of the cross-bars with copper wires of computed ordinate lengths.

When all concrete was deposited in the rib forms (except in the key spaces) the total deflection of the cables, due to the load, did not exceed by $\frac{1}{2}$ in. their computed deflection (*Fig.* 6).

Adjusting the intrados curvature to the true curvature was effected by adjusting so that the plumb-bobs were brought to the same elevation. It was found only necessary to tighten up the nuts under the two central cross-bars ½ in. and to slack off the nuts under the extreme end bars ⅜ in. Twelve hours after depositing the concrete in the key spaces the suspended centering was dropped, or "struck," simply by slackening off both cables.

Judging from experience the striking of suspended centering is accomplished with such ease and uniformity that the usual dangers arising from shock and jar to the completed arch appear to be eliminated.

This, and the further fact that all the concrete in the arch rib is set up at the time of filling the key spaces, made possible the striking of the suspended centering of the 53 ft. Portsmouth arch after 12 hours, and the Chickahominy River 48 hours after keying.

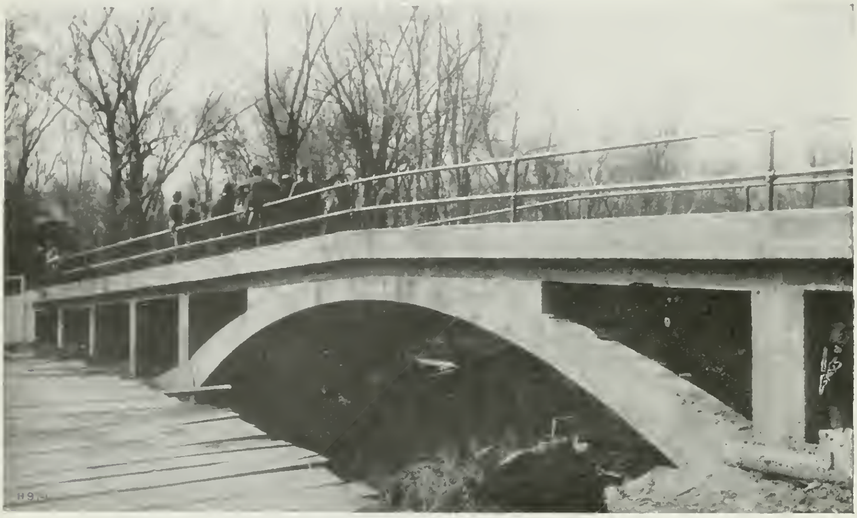


Fig. 11. View of a portion of completed bridge.
Chickahominy River Bridge.

THE CHICKAHOMINY RIVER BRIDGE.

Its Design and Construction without Falsework by means of Suspended Flexible Centering.

The design of the Chickahominy Bridge as built provides for a reinforced concrete structure, composed of a three-rib arch, having a clear span of 50 ft., the approaches being of beam and slab construction in panels of 13 ft. c. to c. of columns.

The length of structure over all is 160 ft., and the clear width of roadway 16 ft. The three arch ribs are similar, each having a depth of 16 in. at the crown and 32 in. at the spring lines, the width of the ribs transversely being 14 in. each.

As the bed of the river was of sand and gravel it was necessary to use batter piles in the arch abutment foundations. The concrete foundation slabs are 1 ft. 4 in. thick (embedding the heads of the batter and plumb piles), and are slightly tapered. The arch ribs spring from skewbacks upon buttresses opposite each. These buttresses are connected by reinforced curtain walls or diaphragms 8 in. thick, located near the skewbacks, and thus act not only as horizontal ties, but also retain the filling deposited in the space between the buttresses.

Cables, Cable Supports, Anchorage, etc.—The cables and centering used in the Portsmouth Arch were ample for this service, but it was desired to obtain cables and suspenders of sufficient capacity for future use upon span lengths up to about 80 ft. For such services $1\frac{1}{32}$ in. diameter galvanised steel cables and $\frac{7}{8}$ in. diameter suspenders were obtained, provided with turnbuckles.

Both approaches were completed prior to the construction of the arch ribs, in order that the cable supports might be mounted upon them, thus affording foundations for same and reducing their height. This arrangement reduced the height of the supports to only 4 ft. 6 in.

The cables were anchored to "dead men" located behind the concrete columns at the ends of the approaches. Slots, or openings, were provided in the roadway floor slab of the approaches, in order that the inside cables might pass through same to their anchorages. The connections between cables and anchorage were made above ground, so that in shifting the cables from one rib to another it was not necessary to make any excavation to the underground anchorage.

The cable supports were constructed of round timber, cut at the site, and timber taken from the old structure. No saddles were used under cables on the supports at either Portsmouth or Chickahominy River. The cables simply rested in notches (lubricated) cut in the supports. Saddles are, of course, necessary in longer spans.

Flexible Centering.—The Chickahominy River flexible centering (*Fig.* 7) was even simpler than that used at Portsmouth. The cross-bars were 3-in. gas pipe 4 ft. long. The ends of the 2 in. × 12 in. wooden longitudinals overlapped on the cross-bars and were notched to receive the bars, thus forming the pivots or points of flexure. The lagging and concrete forms were nailed on top edges of longitudinal members.

Adjustment for Elevation and Curvature.—After the cables were loaded by the concrete (wet) rib sections only 1 in. vertical adjustment for curvature was necessary, as determined by the plumb-bobs. The key spaces were then filled (*Figs.* 8 and 9.).

The maintenance of proper elevation, curvature, etc., being subject to such complete control, the intrados curvature upon completion was perfect in every rib. As stated, the cables and suspenders used were capable of constructing spans of 80 ft. and under. Simply by substituting differently curved fillers, or furring members, upon the longitudinals, and adding or decreasing panels of flexible centering, the plant is ready for use in constructing spans of any radius and length within the capacity of cables and suspenders used.

In order to assure perfect control at all times—in case of freshets—the main cable turnbuckles were located above the floor slab of the approaches. This location also afforded a full view of the entire ribs, intrados, suspenders, etc., to the operator at the main cable turnbuckle levers when making adjustments or "striking centres."

As soon as possible after completing a rib the forms were stripped off, in order to shorten the setting time of the concrete rib, sections, and keys. Forty-eight hours after the completion of the first rib the flexible centering was struck and transferred to the second rib—adjacent—as described, and thence to the third rib (*Fig.* 10).

After completion of the ribs and removal of cables the spandrels, beams, and floor slabs were built and the structure completed (*Fig.* 11). The spandrels were anchored to the arch ribs by $\frac{3}{4}$-in. radial bars, as shown.

Prior to acceptance the Commissioners required the specified test to be made under the supervision of their engineers. The specified test load was a 12-ton road roller, which was operated at various speeds back and forth over the entire roadway area between kerbs.

The work was executed by Mr. Charles McDermott, contractor, of Washington, D.C. The structure was designed by the writer, and submitted to the Highway Commission for revision and approval. Mr. Wilbur J. Watson, of Cleveland, Ohio, acted as consulting engineer.

STEEL FRAME CONSTRUCTION v. REINFORCED CONCRETE.

We frequently hear ironmasters regretting the advance of reinforced concrete and the superseding of steel frame construction, but as a matter of fact many ironmasters have now started rolling the sections of bars for reinforced concrete contractors, and the introduction of the new material has only meant a change in the class of goods turned out at their works. In Germany the ironmasters have very unwisely thought fit to attack reinforced concrete, and the following trenchant reply of the German Concrete Association is most instructive reading. —ED.

At a meeting of the Association of German Ironmasters, held at Düsseldorf, Herr Fischmann, representing the Steel Union, read a paper on the use of steel in building. The greater part of this paper was devoted to a comparison of construction in steel and in reinforced concrete, greatly to the disadvantage of the latter. The Council of the German Concrete Association, considering that the paper in question does not present fairly the relative qualities of the two systems, has now issued a formal reply, the text of which has appeared in various German technical papers.

Herr Fischmann attributes the decline in the output of steel sections, from 1,200,000 tons in 1906 to 830,000 in 1908, in part to the development of reinforced concrete. It is pointed out, however, that the change is really due to market fluctuations, the output in 1909 having again risen to 1,045,000 tons. There is an admitted increase in the output of steel rods used as reinforcement, and if this has meant a decline in the use of rolled joists the result is a transfer, but not a diminution, of the production. This will be realised on considering the number of purposes now fulfilled by reinforced concrete, and thus causing a demand for steel, for which steel was not formerly used at all. Such, for instance, are foundations, rafts, piles, wells, retaining walls, embankments, silos, loading stages and inclined planes, external walls and roofs, reservoirs of all kinds, canals and aqueducts, fortifications, dams, chimneys, monumental towers, reinforced concrete bridges as substitutes for masonry or massive concrete, etc. The progress of reinforced concrete, therefore, although it may react unfavourably on the production of joists, cannot but be beneficial to the steel industry as a whole.

The increase in the use of reinforced concrete is not merely due to its cheapness, but to a much greater extent to the actual advantages which it possesses over steel. The first of these is its greater safety when exposed to fire or to corrosive influences. Whilst unprotected steel expands and softens under the influence of great heat, thus becoming actually a destructive agent, steel properly embedded in cement concrete is to be regarded as a true fire-resisting material, hence its advantage in the construction of houses, schools, theatres, factories, warehouses, etc. The same security is given against rust, thereby preserving the steel and saving the expense of repeated repainting.

The monolithic character of a reinforced concrete structure is not only a guarantee of stability, but also facilitates erection, from the fact that the entire structure is erected in the same manner by the same contractor, and the delays and friction inevitable when a number of contractors share the building, as when the materials are steel and brick, are avoided. The great adaptability of reinforced concrete is a further advantage, and gives greater scope to the artistic architect than does steel skeleton construction, a suitable architectural style for which has not yet been evolved.

Herr Fischmann remarks that the statical computation of reinforced concrete structures is still very obscure. The reply is made that certain difficulties still remain in both systems of construction, but that the vast amount of experimental work completed or in progress in the great testing stations actually places reinforced concrete in a superior position to steel, the introduction of which antedates the founding of these laboratories, so that its statical rules were necessarily arrived at on theoretical grounds.

How steel construction suffers from its lack of experimental work is seen from Prof. Bach's experiments on U-beams, published in 1909 and 1910, from which it appears that the strength of such beams fell in all cases below that calculated to the extent of from 8 to 34 per cent. in the case of loading in the plane of the principal vertical axis. The factor of safety of reinforced concrete structures actually increases in the course of years, owing to the progressive hardening of the concrete. No case is known in which a structure properly erected in accordance with the official method of computation has proved insufficiently strong. The monolithic character of such structures gives them an exceptional power of adaptation to dynamically varying stress.

The objection is also raised that the construction of a reinforced concrete building in accordance with the design is very difficult. This is not the case, and accidental variations in the placing of the reinforcing rods still leave a wide margin of safety. The error from this source does not exceed that due, in steel construction, to the eccentric placing of flanges, effects due to torsion, and the like. The custom of neglecting the tensile strength of the concrete is a further safeguard. The strange remark is made by Herr Fischmann that should the reinforcing rods happen to be, for example, 10 mm. in diameter instead of 11 mm., the quantity of metal introduced would be insufficient. It would, however, be remarkable if a modern rolling-mill were to send out material so variable in size. Actually, owing to the wear of the rolls, rods are commonly slightly thicker than their nominal section.

The variability of concrete is a further objection raised against it. It is true that concrete from two different sources may differ widely in properties, but the experimental work already accomplished has made it perfectly possible to determine what will be the strength of a concrete prepared from given materials under given conditions. By the German regulations the contractor is bound to satisfy the building authority that the materials and mode of mixing he employs actually yield concrete of the strength required by means of tests on 28-day blocks, or, when this would cause too long a delay, by tests after 14 days, the 28-day tests being carried out subsequently as a confirmation. Small variations in the strength of the concrete are negligible in view of the fact that the compressive stress allowed is only one-sixth of the breaking stress. In steel under tension, whether in steel structures or in reinforced concrete, the determining quantity is the elastic limit, and this is only two-and-a-half to three times the stress allowed. Further, a building is not usually loaded until more than 28 days after its erection, and the strength of the concrete is then greater than that indicated by the tests.

The comparison made by the reader of the paper is between badly constructed reinforced concrete work and well constructed steelwork. This is not a fair method of comparison. Disasters due to bad work or faulty design have occurred in recent years in structures erected according to both systems, but are not to be attributed to inherent defects of those systems.

Herr Fischmann attributed a part of the progress of reinforced concrete to the fact that it is a novelty. This is contrary to all experience. In no industry is there more conservatism than in building. This is easily conceivable when we remember that buildings represent the sinking of large capital, and that they are intended to be permanent. Builders and owners are therefore naturally reluctant to use any but well-known materials or methods, or to adopt any novelty the use of which may be attended by risk.

As a matter of fact, concludes the Council of the German Concrete Association, reinforced concrete construction has had to contend with many prejudices and with much opposition, and if, in spite of this, its progress is a triumphal one it owes it in the first place to the simple and rational principles underlying it and to the technical and commercial advantages which result therefrom.

SOME RECENT WORKS IN REINFORCED CONCRETE.

By G. C. WORKMAN (of Messrs. Edmond Coignet, Ltd.).

At a meeting of the Concrete Institute held on December 14th last, Mr. G. C. Workman read a paper entitled "Some Recent Works in Reinforced Concrete," and illustrated same by some very interesting lantern slides. A summary of the paper appears below. Most of the works have already been described at length in our journal.—ED.

The object of this paper was to describe a few of the numerous works recently executed in reinforced concrete, and to bring out some of the more interesting features of each particular case and to show how various difficulties have been overcome.

Owing to modern requirements, there is a growing tendency for the various branches of engineering to become specialised in order to obtain, by a continual study of each subject, the maximum amount of economy and efficiency. For the past ten years the author had been identified exclusively with the "Coignet System," which was one of the first rational and scientific methods of reinforcing concrete. As there are now a certain number of different ways of reinforcing concrete, he would give a short description of the methods employed in the various works which were about to be described. Commencing with the beams, which are usually considered the most important part of a structure, round bars of mild steel are used, the diameter varying between $\frac{1}{2}$ in. and $1\frac{5}{8}$ in. The steel frameworks are designed in such a manner as to form independent units which can be easily made on the site and placed on the centering when required. In some cases the tension bars for the lower portion of the beams are all straight with a corresponding parallel bar in the upper compressed portion of the beam; a mechanical bond between the upper and lower bars is provided by means of stirrups made of round mild steel wire varying between $\frac{3}{16}$-in. and $\frac{1}{4}$-in. diameter. These stirrups have their ends bent over the top bar, and they are fastened by means of annealed wire to the main bars so that their position cannot be changed during the process of concreting. The spacing of stirrups is, of course, varied in accordance with the shearing stresses.

Another method of preparing these steel frameworks consists in grouping together seven tension bars of comparatively small diameter; six of these bars are bent upwards at an angle of forty-five degrees and hooked over a longitudinal top bar $\frac{1}{2}$ in. diameter in such a manner as to form a kind of truss. Additional stirrups are also provided, as in the first method described, to counteract any extra shear.

This last arrangement has the advantage of being more economical than the first one, because the section of steel in tension is made to decrease gradually towards the points of support where the bending moments are less, the extra steel being used to resist shear or diagonal tension.

In regard to shear, he would point out that in the first method, comprising vertical stirrups and horizontal bars, the vertical shear is taken up by the main bars and the horizontal shear by the vertical stirrups, whereas in the last-mentioned alternative the vertical and horizontal shear, or, in other words, the diagonal tension, is counteracted by the bars bent at an angle of forty-five degrees.

The longitudinal top bar is, of course, necessary for the preparation of each unit. This bar is very useful to suspend the framework in the centering, and as it is situated in the upper portion of the beam it increases the resistance to compression. The upper end of the stirrups and shear members is secured by means of hooks to the top bar. This precaution is in accordance with the new rules of the Joint Committee of the Royal Institute of British Architects, which state that in every case care should be taken to bend or otherwise secure the ends of the rods to prevent them from slipping in the concrete. The negative bending moments in beams are counteracted by joint bars of sufficient length.

In many of the buildings the steel frames of the pillars are made horizontally and placed in position in the casings when required. These frames are composed of main bars bound together either by $\frac{3}{16}$-in. or $\frac{1}{4}$-in. wire, forming spirals or horizontal ties with a pitch of about 6 in. The object of these ties is not so much for strength as to keep the vertical bars in their proper position during the concreting.

Concerning the reinforcement of rectangular floor slabs, this is provided by simply placing principal bars of about $\frac{1}{2}$-in. to $\frac{5}{8}$-in. diameter in the smaller spans and $\frac{1}{4}$-in. distributing rods at right angles to the others, so as to form a mesh, the bars of which are bent up gradually over the beams in order to resist contraflexure.

As previously mentioned, round bars have been used throughout because they have the advantage of being easily procurable in the open market, and because this section produces the minimum amount of contact between the various bars forming the units. It is obvious that flat surfaces of contact between bars should as much as possible be avoided because rust is likely to develop in such places, and this defect may ultimately become a source of danger. Another reason for round bars being favoured is that they offer a better surface of adhesion than any other available section.

Having briefly described the general principles of the reinforcement used in the buildings dealt with below, the lecturer put before them a few diagrams showing the two methods of making steel frames for beams already described.

Tobacco Warehouse, Bristol Docks.—One of the first large buildings with which he had to deal was the second tobacco warehouse for the Bristol Docks.

The general dimensions of the structure are: 200 ft. in length by 102 ft. in width and about 96 ft. in height, and there are altogether nine floors calculated for a superload of $1\frac{1}{2}$ cwt. per square foot.

This building is identical in appearance to the first tobacco warehouse, but the inside is in reinforced concrete instead of being in steelwork with concrete floors.

Besides the advantages of fire resistance and economy which this building offers over the first one, the internal accommodation is larger on account of the walls being only about 14 in. in thickness throughout the height of the building, instead of the considerable thickness of brick walls required in the first building. These thin brick walls or panels were supported at each floor by lintels, and the building has been divided in two by a partition in reinforced concrete 6 in. thick.

The various floors were arranged in such a manner as to minimise the risk of fire by cutting off any particular floor from the others. The level of the floors is slightly raised at all lift openings and doors, so that it would be possible to flood any particular floor with a couple of inches of water if required.

This warehouse has been erected on reinforced concrete piles varying in length between 40 ft. and 50 ft.; 650 piles were required for the foundations. The load coming on each of the internal pillars amounted to 300 tons, and it was necessary, in order to transmit this load on to the ground, to group six piles together, uniting the heads by means of a cap.

Each pile was calculated to stand a safe load of 56 tons, and the diameters of the piles varied between 14 in. and 15 in. The piles were driven into the ground by means of a steam monkey weighing 2 tons. The particular pile shown on the screen took about three-quarters of an hour to drive to its final set at a depth of about 45 ft.

Buildings at Rainham, Essex.—*Fig.* 1 is a photograph of one of several buildings erected at Rainham, Essex, for Messrs. J. C. and J. Field, Ltd. The site is situated on the riverside and it is extremely marshy, so that the buildings had to be constructed on spread foundations or rafts in reinforced concrete. Reinforced concrete buildings, being monolithic and lighter than brick or masonry work, are very well adapted to cases where unequal settlement is likely to occur, especially when such buildings are erected upon a reinforced concrete raft.

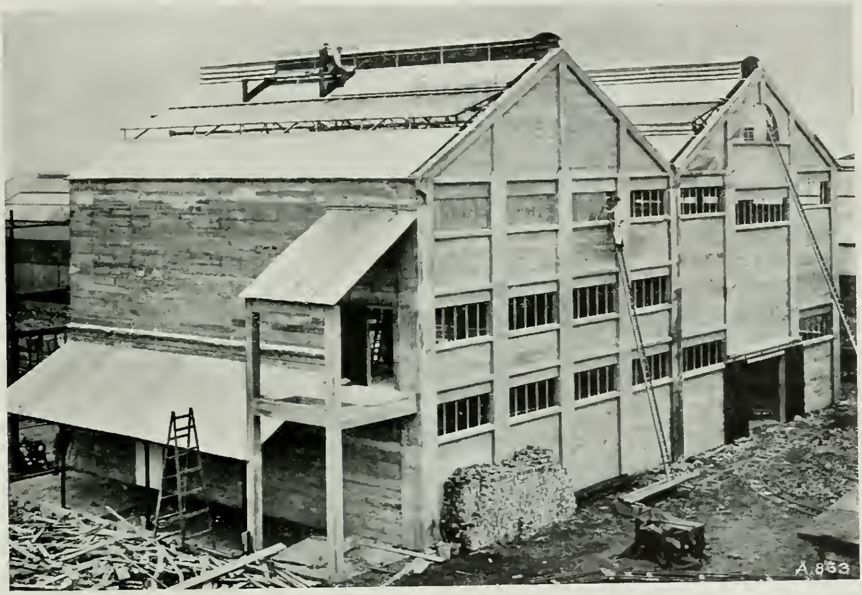
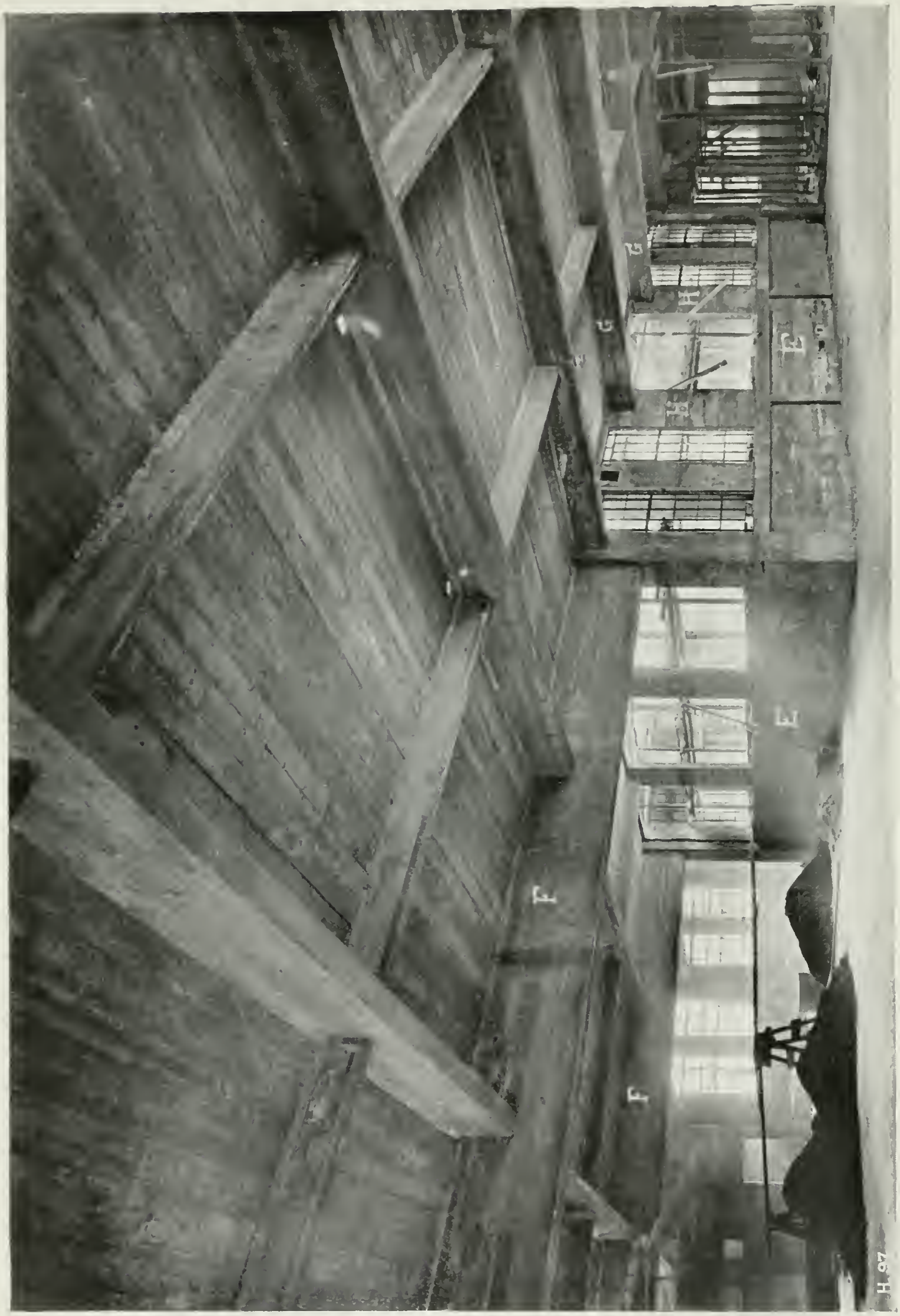


Fig 1. Reinforced Concrete Warehouses at Rainham.

Two Other Examples.—A perspective drawing of extensive premises erected in the city for Messrs. J. Grossmith, Son and Co., perfumers, was shown. The building has altogether eight floors, the interior and roof being entirely in reinforced concrete. An interesting feature of the building is that the heavy front and side lintels are supporting the whole of the brickwork above.

A photograph was shown of the flooring, during construction, for the hostel for women students at the Birmingham University. The total area of flooring was approximately 13,000 sq. ft. These floors were tested with a load 50 per cent. in excess of the specified superload, and the deflection did not exceed a thousandth part of the span.

There seemed to be a considerable difference of opinion between the various methods of stopping the concreting in floor slabs, in beams, and in pillars. Personally, he was of opinion that the best method of stopping the concrete of the floor slabs is to stop



Interior View of First Floor.
Fig. 2. The New Western District Post Office.

the work over the beams, also that the concreting of beams should be stopped over the points of support.

Concerning the pillars, the footings should be made first and the body of the pillar carried up to the level of the soffit of the beams, the remaining portion being concreted at the same time as the beams.

New Western District Post Office.—One of the most interesting buildings with which he had been associated was the new Western District Post Office, which was erected about two years ago under the supervision of H.M. Office of Works.

This building comprises a block of 200 ft. frontage and 160 ft. in depth, with basement, ground floor, first and second floors, and flat roof.

One of the chief features of the building is that the ends of the principal beams carrying respectively the first floor, second floor, and roof are suspended bodily by means of strong steel stirrups to upper beams 6 ft. high, forming the internal walls surrounding the inner court above the various floor levels. By this means it was possible to obtain a maximum height for the windows, which was a condition stipulated by the Office of Works. This condition, however, brought about a question as to whether this method of suspension could be used without danger, the two beams having a span of 39 ft. and supporting the first floor with a superload of 1 cwt. per sq. ft., the upper supporting beam having a span of 45 ft. There was no evidence that this particular arrangement had ever been adopted before, at least for spans of such large dimension. The problem was solved in a satisfactory manner, with the result that the two suspended beams, 39 ft. long each, and the beams across the inner court supporting the skylight, appear to have been made in one single span of about 118 ft.

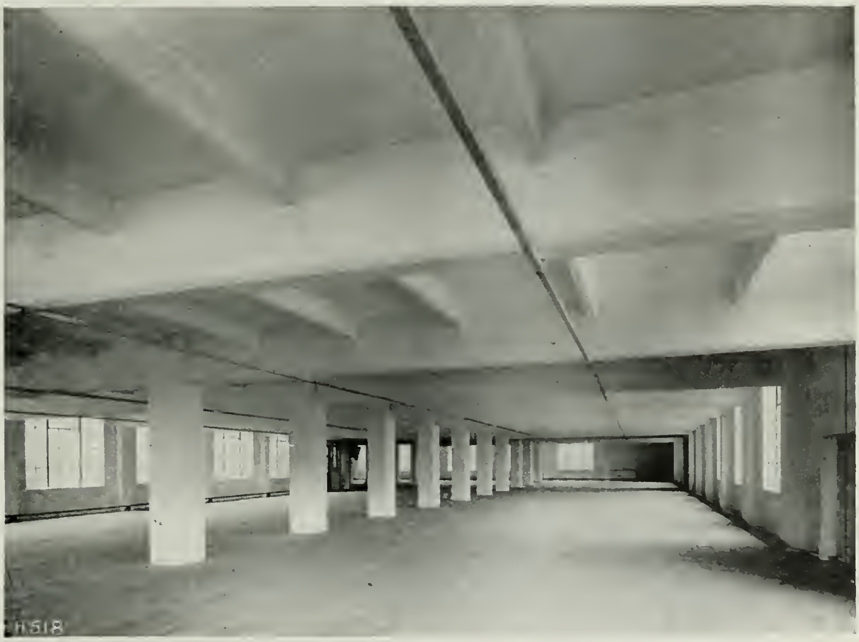


Fig 3. View of Interior.
FACTORY AT BAILIFFE BRIDGE, YORKS.

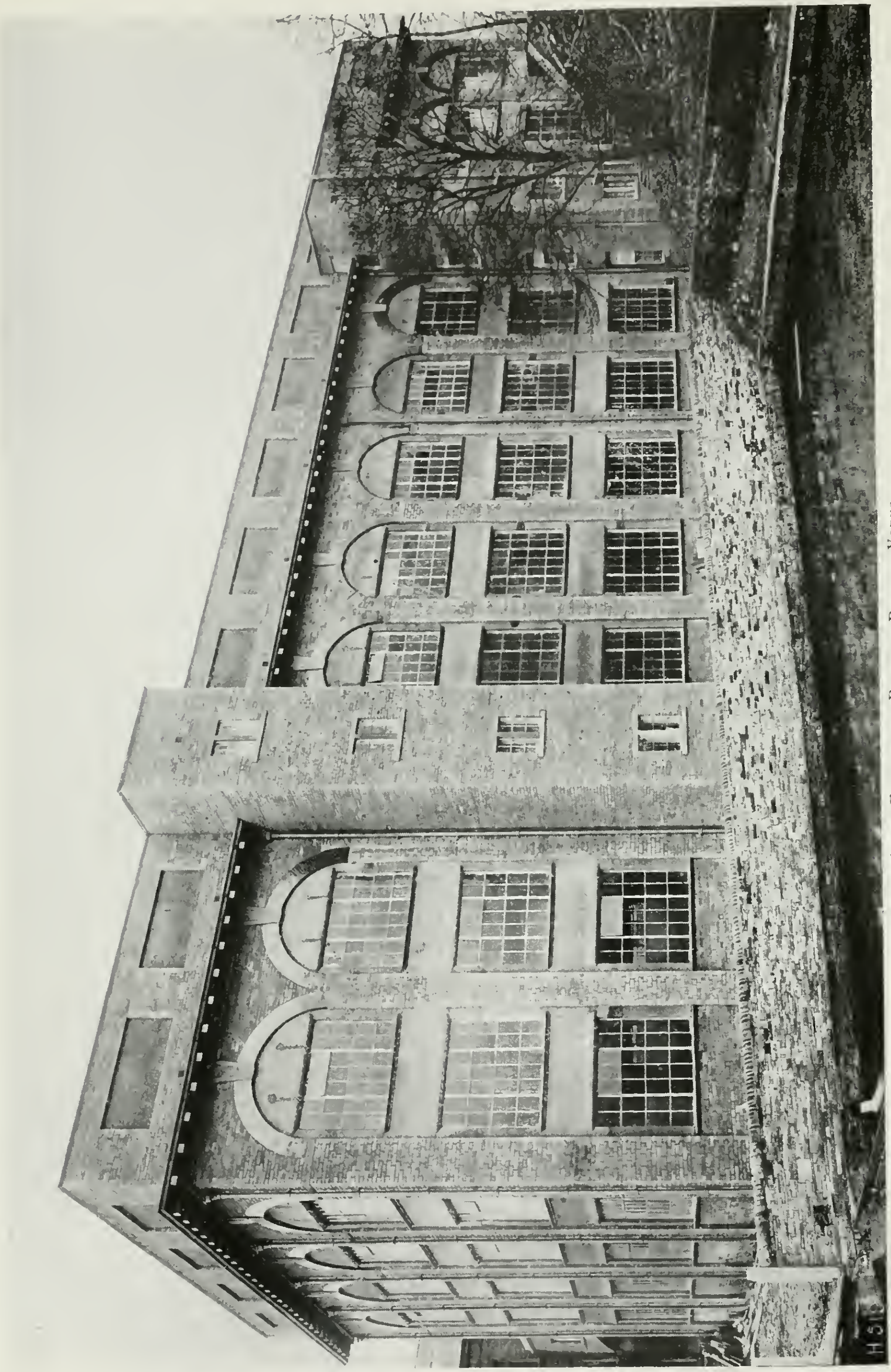


Fig. 4. Factory at Bailiffe Bridge, Yorks.

Another noticeable feature is found in the extensive beams, over 12 ft. in height, which carry the other end of the suspended beams. These 12-ft. beams have a span of 45 ft., and it was necessary to provide in the portion of these beams underneath the first floor a series of large windows situated below the neutral axis of the beam, which occurs approximately at the level of the first floor.

Fig 2 is a view of the first floor, showing the extensive area of flooring without any pillars.

Factory at Bailiffe Bridge, Yorks.—*Fig.* 4 is a view of a large factory erected about eighteen months ago at Bailiffe Bridge, near Leeds, for Messrs. T. F. Firth and Sons, Ltd., carpet manufacturers.

The main building is four storeys high and 176 ft. long, with a width at one end of 70 ft. and at the other end of 111 ft., the building being in the shape of an "L." The total floor area is approximately 52,000 sq. ft. The factory is built on a very steep incline, and to overcome difficulties of levels on the site from the ground to first floor a retaining wall 16 ft. high was constructed in reinforced concrete. The staircases throughout the building are also in this material.

Fig. 3 shows the interior of one of the floors with a central row of pillars, the span of the main beams being 35 ft. It will be noticed that the main beams have been arranged parallel to the windows in order to get the maximum amount of light for weaving purposes.

Money Order Office, Holloway. — *Fig.* 5 is the front elevation in reinforced concrete of the new Holloway Money Order Department, which is the latest of a series of large buildings for the extension of post-office facilities in London, under the instructions of H.M. Office of Works.

The entire construction, with the exception of the front wall elevation, is in reinforced concrete. The building is in the shape of an "E." The total length of the front measures approximately 292 ft. and the depth of the main body about 50 ft. The three wings measure respectively 88 ft. by 42 ft., 101 ft. by 42 ft., and 76 ft. by 42 ft.

The building is composed of a basement, five reinforced concrete floors, and a flat roof in the same material. There are also a large number of intermediate floors in the main body, which are used for cloak-rooms. The total height of the structure from the ground level to the roof is approximately 85 ft., and all the stairs and balconies are in reinforced concrete.

The superload on the various floors was only $\frac{3}{4}$ cwt. per sq. ft., and the floors were tested by means of piling up bricks to a sufficient height to produce a superload of about $1\frac{1}{4}$ cwt. per sq. ft. There was no permanent deflection, and the results of the tests were considered satisfactory.

Whilst dealing with the question of superloads, he would like to mention that it has been his experience that architects and engineers are inclined to specify superloads which are obviously far more considerable than what is really necessary. This is probably due to the fact that in many cases reinforced concrete is a novelty to them, and by specifying a comparatively high superload they hope to guard themselves against failure of the work. It is well established, however, that any failures which have occurred have been due either to the premature removal of props and centering, or, in a few isolated cases, to faulty design, so that in reality nothing is gained in security by specifying a higher superload than is required.

In the case of this building the Office of Works, who are thoroughly acquainted with the use of reinforced concrete, have not hesitated to specify a superload of $\frac{3}{4}$ cwt. per sq. ft., which is quite ample for their requirements, and which has the advantage, of course, of being economical.

Government Buildings, Jamaica.—A very large amount of reinforced concrete work has been carried out during the last two years in Jamaica.

Fig. 6 is a photograph of the new Government buildings erected at Kingston, Jamaica, for the Government.

This large building, which is entirely in reinforced concrete, contains the General Post Office and the Treasury. The dimensions of the Post Office building are 121 ft. by 132 ft., and the Treasury 187 ft. by 88 ft.

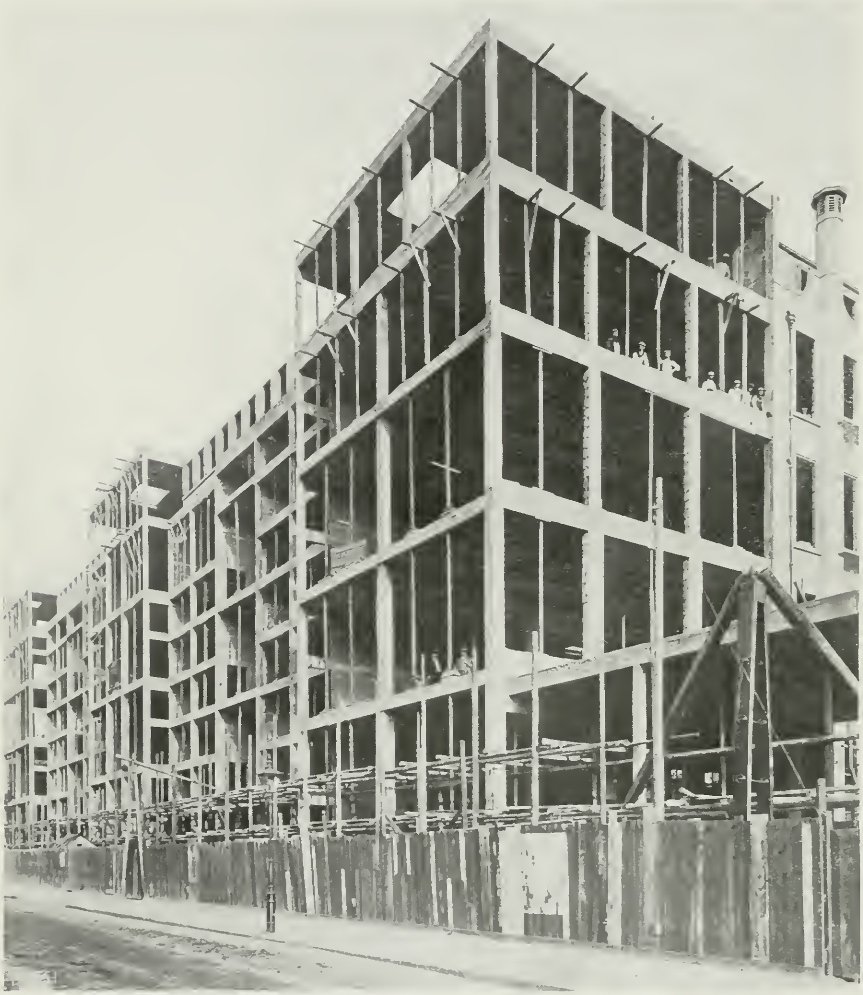


Fig. 5. Front elevation in reinforced concrete, ready to receive the masonry walls.
THE NEW MONEY ORDER OFFICE, HOLLOWAY.

In preparing the design the architects had to consider the question of earthquakes, and consequently the foundations of these buildings have been constructed in such a manner as to offer the least possible obstruction to shocks or movements of the ground. The foundation is practically in the form of an immense raft composed of strong slabs and beams, the latter uniting the various footings of the pillars.

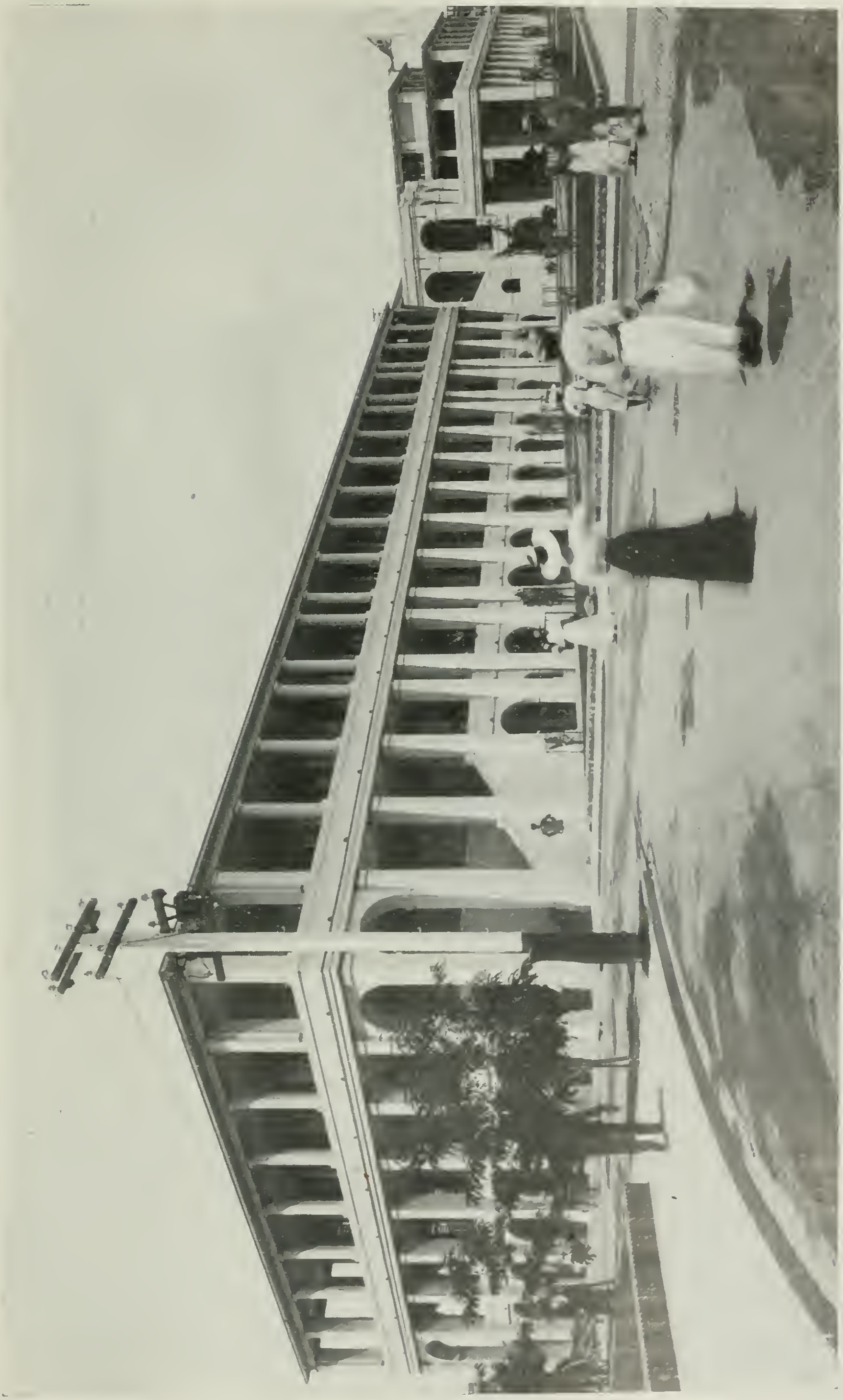


Fig. 6. Post Office and Treasury Buildings, Kingston, Jamaica.

The superstructure is of the type usually adopted in hot climates, with spacious verandahs and colonnades. The scantlings of the pillars vary between 24 in. and 18 in., and some of these are octagonal in shape.

The average span of the beams supporting the roof is 20 ft. and the average thickness of the slabs is 4 in. The superload on first and second floors is 150 lb. per sq. ft., and the roof was calculated for 84 lb. per sq. ft.

The new King's House at Kingston, Jamaica, was carried out simultaneously with the one previously described. The foundations were also made by means of a raft, and, in fact, the same type of construction was adopted as for the public buildings.

Various Other Structures.—The lecturer showed a photograph of a foot-bridge erected on the Thames bank at Erith for the Erith Oil Works, Ltd. The total length of the work is approximately 150 ft., with a width of about 6 ft.

Another illustration was shown of a foot-bridge at Higham-Hellesdon, the total length of which is approximately 340 ft., with a width of 7 ft. It was built on fifty piles 13 ft. long each.

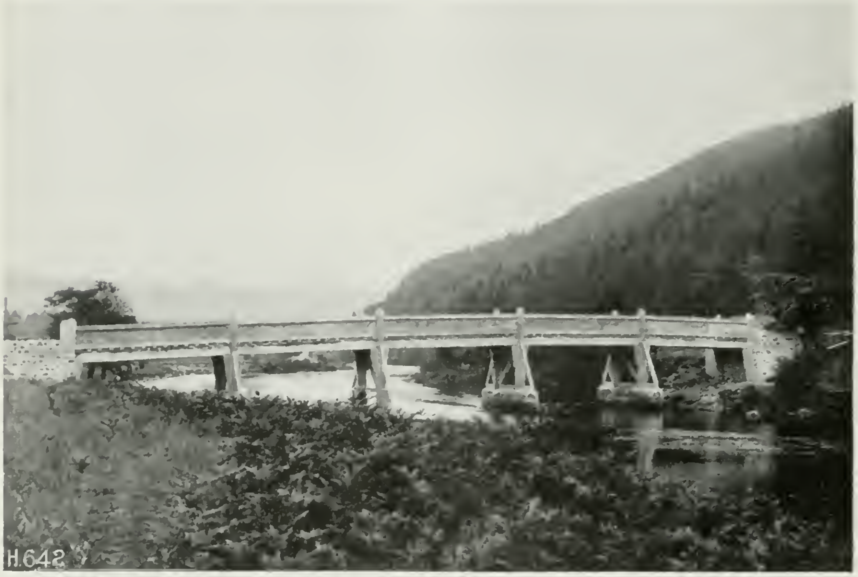


Fig. 7. View of finished bridge.
REINFORCED CONCRETE BRIDGE AT STRUY-BY-BEAULY.

Fig. 7 is a view of a road-bridge erected at Mauld, near Inverness. The total length of the bridge is 180 ft., with a width of 12 ft. between parapets. Four of the spans measure 35 ft., and the remaining span measures 45 ft. The foundations of the piers were made by driving a certain number of reinforced concrete piles in the bed of the river, encasing these in a solid block of concrete.

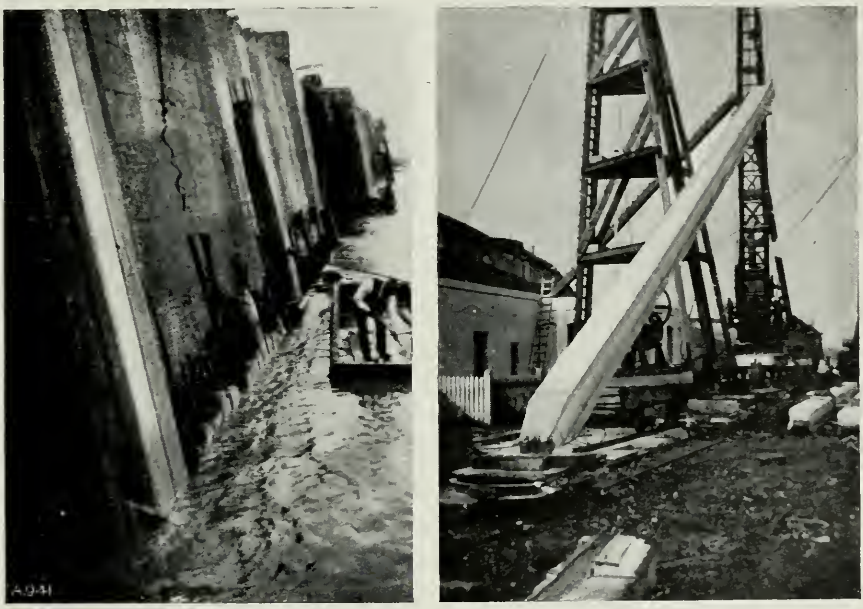
Another bridge shown was one of the two erected for the Powell Duffryn Steam Coal Co., Ltd., under the supervision of their engineer, Mr. J. M. Greenhow.

This bridge, which is of the bow-string type, is calculated to carry two lines of railway with locomotives weighing 50 tons and wagons 19 tons each. The bridge is built on the skew, the dimensions being 56 ft. 9 in. in span, with a width of 26 ft. 8 in. and a height of bow-string beam of 13 ft. 3 in.

The remarkable feature of this bridge is that it has been constructed underneath and on both sides of an existing steel bridge, which had become unsafe owing to corrosion produced by the fumes of sulphur and ammonia. The advantages of reinforced concrete in such cases are obvious. The reinforced concrete bridge was first allowed a sufficient time to harden and was then ready for service on the removal of the existing steel bridge, which operation was carried out in forty-eight hours.

An elevated tank was also erected for the same mining company. The general dimensions of the tank are 100 ft. long, 32 ft. wide, and 5 ft. 3 in. high.

The remarkable feature of this tank is that although the inside was not rendered it is perfectly watertight. This is due to the fact that the aggregate was very carefully graded, and also on account of the height of the water being only 5 ft.



Side of Pier. A Pile.

Fig. 8. Reinforced Concrete Underpinning of the West Pier, Newhaven.

A photograph showing the underside of the large bunker erected at Kinlochleven for the British Aluminium Co., Ltd., was put on the screen.

The total capacity of this large bunker was over 80,000 cu. ft. The main feature of this enormous work is the suspended hopper, which has been calculated to carry a total load of about 2,000 tons in addition to its own dead load.

This bunker is divided up into a certain number of transverse compartments forming large suspension beams, carrying the horizontal beams and inclined panels of the hoppers. In this manner the entire space underneath the bunkers is free for conveyors and other machinery.

Fig. 8 is a view of the West Pier, Newhaven, which has been underpinned by means of about 700 reinforced concrete piles.

It was found necessary to replace the old wooden piles supporting the concrete work on account of the decayed condition of the wood. The reinforced concrete piles—or, more accurately, sheet piles—were 16 in. by 16 in. in section, and some of the longest piles measure 52 ft. over all. They were driven by means of a steam monkey weighing 2 tons. Some of the piles were driven a considerable distance into the hard chalk, which is a proof of the strength of reinforced concrete piles.

Another view shown was of a wharf recently constructed at Portslade. The work is composed of piles and sheet piles connected by means of beams and supporting reinforced concrete panels held in position by means of tie beams and concrete anchor blocks.

The main piles are 12 in. by 12 in., with a length of 25 ft. These piles were driven 15 ft. centres, the intervening space being made up with sheet piles 22 ft. long, having a sectional area of 6 in. by 18 in. The remarkable feature of this work is that the piles and sheet piles had to stand very heavy blows owing to an exceedingly hard stratum of ground through which they had to be driven.

He had endeavoured in the various examples of work placed before them to illustrate all the various uses of reinforced concrete—namely, for floors, beams, pillars and walls, spread foundations, also foundations by means of piles, bridges, bunkers, water-tanks, and finally marine works.

He would now conclude by saying that the object of this paper had been to show how certain practical difficulties might be solved in reinforced concrete construction, and it was his hope that some of the works put before them that night would still further strengthen the confidence which English architects and engineers are showing in this comparatively new method of construction.

SOME RECENT WORK ON THE ALTERATION IN VOLUME OF CEMENT AND CONCRETE.

By PERCY C. H. WEST.

In our May, 1911, issue we published an article by Mr. Percy West on "The Dilatation of Cement and Concrete," and below we present a further article by him dealing with this subject, which claims attention.—ED.

In a recent issue a review of the work published with reference to the expansion and contraction of cement and concrete was given. Since then, however, some further matter of an extremely interesting character relating to this important question has been made public by Mr. A. S. Goldbeck, of the United States Office of Public Roads. This work possesses an especial value, in that concrete test-pieces were used instead of the Standard Mortar and Neat Cement test-pieces employed by most other investigators. The materials of which the concretes were prepared consisted of Portland cement, river sand, and crusher-run Gneiss. The test-pieces were of square section, of 8-in. side, and were 5 ft. long. In these prisms iron rods, 11 in. long by ½ in. diameter, were embedded 50 in. apart. To these rods, rounded conical plugs were secured, and the measurements were taken from these fixed points by means of a Brown and Sharp micrometer, held at one end of a rectangular frame formed of two steel bars, ¼ in. in diameter, connected by means of square-sectioned yoke-pieces.

The initial readings were taken as soon as the bars were sufficiently hard to be removed from the moulds. The measurements were in all cases made between a temperature of 20° and 24° Centigrade, a thermometer being inserted in a hole moulded in the test-pieces and another hung alongside. The slight variations in temperature were compensated for, and the readings reduced to 20°, Professor Norton's coefficient of 0·0000099 being used. As previously noticed, this figure agrees closely with the results of Keller and Pence.

Mixtures gauged with different percentages of water were tested, those classed as "Very Dry" being mixed with 8·5 per cent. and the wet mixtures with 10-12 per cent. of water.

The results obtained, when plotted, gave fairly smooth curves with but few minor irregularities. From these smoothed curves the following figures are derived. The test-pieces exposed to air in all cases showed a contraction, the rate of contraction decreasing with age until at three months it became very small. It was found that the dry mixtures showed a greater contraction than did the wetter mixtures. It must, however, be remarked that the differences observed were slight.

Contraction of Concrete Exposed to Air (Goldbeck).

Mixture.	1 : 2 : 4 Dry.	1 : 2 : 4 Wet.	1 : 3 : 6 Dry.	1 : 3 : 6 Wet.
10 days ...	0·00015	0·00010	0·00010	0·00010
15 ,, ...	0·00021	0·00015	0·00016	0·00015
20 ,, ...	0·00025	0·00021	0·00027	0·00020
30 ,, ...	0·00035	0·00030	0·00038	0·00027
60 ,, ...	0·00048	0·00045	0·00052	0·00041
90 ,, ...	0·00052	n.d.	n.d.	n.d.
120 ,, ...	0·00052	n.d.	n.d.	n.d.

Schüle (1909) has published a lengthy series of tests of a similar type extending over a period of one and a half years, and it is of interest to compare his results with those obtained by Goldbeck. The concretes tested by Schüle were composed of 150 kg. of cement to one cu. metre of aggregate, roughly 1 : 10; 300 kg. of cement to one cu. metre, 1 : 5; and 450 kg. of cement to one cu. metre, 1 : 3·3. The results are tabulated below. As will be observed, there are a number of minor irregularities, but by plotting the results the general course of the contraction can be followed. There is a fairly sharp decrease in length up to the age of three months, after which the rate of decrease is much slower, and at between four and six months the length remains nearly constant.

Contraction of Concrete Exposed to Air (Schüle).

Composition.	450 kg. Cb. M.		300 kg. Cb. M.		150 kg. Cb. M.	
Cement.	Z.	V.	Z.	V.	Z.	V.
4 days	0·000135	0·000003	0·000012	0·000063	0·000105	0·000042
7 ,,	0·000054	0·000111	0·000093	0·000105	0·000195	0·000123
28 ,,	0·000465	0·000390	0·000357	0·000330	0·000378	0·000351
84 ,,	0·000453	0·000390	0·000342	0·000312	0·000393	0·000369
210 ,,	0·000426	0·000393	0·000327	0·000321	0·000285	0·000309
1 year	0·000591	0·000558	0·000420	0·000429	0·000414	0·000414
1½ ,,	0·000543	0·000504	0·000408	0·000378	0·000417	0·000402

Schüle's results are in very fair agreement with those obtained by the Ecole des Ponts et Chaussées, and to which reference was made in the previous article. Correcting the figures there given, for alteration in length due to change of temperature, the coefficient 0·0000099 being used, we get :

3 days	12 days	30 days	47 days	93 days	147 days
0·00000	0·00011	0·00021	0·00022	0·00034	0·00039

These results were obtained with a mixture of 300 kg. of Portland cement to 0·4 cu. metres of Seine sand passing the 5 mm. screen, and 0·8 cu. metres of shingle passing the 25 mm. screen. Schüle's aggregate was composed of one part of sand to two parts of stone. The stone was from 8 to 12 mm. in diameter and the sand averaged 2 mm. The compositions of the concretes are not strictly comparable, but in actual work a number of factors tend to affect the character of the concrete : minor variations in the grading of the stone, variations in the amount of water employed in the mixing being two of many. The strict application of an extremely accurate figure obtained from laboratory work is, therefore, probably of little more value than an approximation arrived at from a study of results obtained from a number of investigators working under slightly different conditions. The following figures are thus derived, and may prove of value in calculations :

CONTRACTION OF CONCRETE EXPOSED TO AIR.

—	1 Month.	3 Months.	1 Year.	1½ Years.
Maximum ...	0·00036	0·00052	0·00052	0·00054
Minimum ...	0·00022	0·00034	0·00040	0·00042
Average ...	0·00029	0·00043	0·00046	0·00048
Difference ...	0·00007	0·00009	0·00006	0·00006

It has already been remarked that the wetter the mixture the lower is the expansion. The poorer concretes exhibit less contraction than do those richer in cement. Goldbeck's 1 : 3 : 6 mixture exhibited only 0·9 times the contraction of the 1 : 2 : 4 mixture. The 150 kg. of cement to one cu. metre of aggregate mixture tested by Schüle gave a contraction of approximately 0·77 of that exhibited by the 450 kg. per cu. metre concrete. It would be expected that the 300 kg. mixture would exhibit a contraction falling between the above, but from the figures given it appears that the contraction is the same as that of the 150 kg. mixture.

THE EXPANSION OF CONCRETE WHEN IMMERSED IN WATER.

Goldbeck determined the effect of keeping the test-pieces continuously moistened during the first fifteen days of the hardening period, and found that in all cases there was an expansion, followed by a contraction on drying out. The contraction at ninety days approached in magnitude that which would have resulted if the test-pieces had been exposed to the air during the whole of the period. In fact, he concluded that the contraction is only delayed by the treatment. Schumann came to a precisely similar result in his early work, and also found, as Goldbeck has done, that on wetting the concrete or mortar there is an expansion followed by a contraction on drying out—a phenomenon observed also with timber, brick and stone.

If the specimen was kept continuously wet, Goldbeck found that the test-piece exceeded its original length. His results do not extend beyond sixty days.

CONCRETE IMMERSED IN WATER (GOLDBECK).

Composition.	Water.	10 Days.	15 Days.	20 Days.	30 Days.	60 Days.
1 : 2 : 4	10–12%	0·00005	0·00005	0·00006	0·00005	0·000025

It is of interest to compare Schüle's results with those given above. As in the case of the results of the same investigator with regard to prisms exposed to air, the work extends over a period of a year and a half. The most obvious feature brought out by these investigations is the marked difference between the expansion exhibited by the 450 kg. mixture and the corresponding figures of the 150 mixture. In the latter case the expansion is only 20 per cent. of the former, whereas with the test-pieces exposed to air the contraction of the poorer mixture was nearly 80 per cent. of that shown by the richer concrete. Another point which is most noticeable is the relatively high expansion exhibited by the cement "Z" in the 300 kg. concrete. In degree this expansion nearly equals that of the 450 kg. mixture.

EXPANSION OF CONCRETE IMMERSED IN WATER (SCHÜLE).

Composition.	450 kgs. Cb. M.		300 kgs. Cb M.		150 kgs. Cb. M.	
Cement.	Z.	V.	Z.	V.	Z.	V.
4 days	0·000018	0·000015	0·000063	0·000048	0·000006	0·000009
7 ,,	0·000054	0·000087	0·000150	0·000069	0·000027	0·000033
28 ,,	0·000096	0·000024	0·000150	0·000057	0·000000	0·000021
84 ,,	0·000171	0·000177	0·000093	0·000093	0·000045	0·000042
210 ,,	0·000276	0·000207	0·000174	0·000111	0·000039	0·000030
1 year	0·000225	0·000186	0·000207	0·000129	0·000042	0·000030
1½ ,,	0·000225	0·000270	0·000201	0·000122	0·000039	0·000015

It is not easy in this case to derive any coefficient which would be of value. No help is obtained by comparing Goldbeck's figures with these; nor from the results of the Ecole des Ponts et Chaussées. Correcting for temperature the figure published by this institution (coefficient 0·0000099 per degree Centigrade), we obtain the following:

30 days	47 days	93 days	149 days
0·000025	0·000025	– 0·00001	– 0·00001

This matter is one upon which more information is needed, and it is to be hoped that some further investigations in this connection will be made in the near future.

THE EFFECT OF REINFORCEMENT ON THE EXPANSION AND CONTRACTION OF CEMENT AND CONCRETE.

So far, only the variation in length of unreinforced test-pieces has been referred to. Goldbeck has published nothing in connection with reinforced specimens. In a previous article the results of the Ecole des Ponts et Chaussées were given, and, corrected for change in temperature, they are repeated below:

CONTRACTION OF CONCRETE TEST-PIECES EXPOSED TO AIR (PARIS).

Concrete 300 kgs. C : 0.4 cb.m S : 0.8 cb.m. Stone.

—	Unreinforced.	Reinforced 2%	Reinforced 2·5%
14 days	0·00011	0·00015	0·00014
30 ,, ...	0·00021	0·00025	0·00021
47 ,, ...	0·00022	0·00025	0·00020
93 ,, ...	0·00034	0·00036	0·00031
150 ,, ...	0·00039	0·00036	0·00041

From these results it would appear that reinforcement had no effect upon the contraction of concrete exposed to air. Entirely otherwise are the results of Schüle, who tested a series of reinforced mortars. He found that the contraction of a 1 : 3 mortar reinforced to the extent of 1·22 to 2·4 per cent. was, roughly speaking, only one-half as great as that exhibited by the unreinforced specimens. The reinforcement appears to have a greater effect on the expansion of test-pieces immersed in water, reducing the expansion in one instance to one-quarter of that exhibited by the unreinforced test-piece, with a reinforcement of 2·4 per cent. The actual figures are given below, and from them it will be observed that the cement "V," with a reinforcement of 1·2 per cent., exhibits practically the same expansion as the unreinforced mortar; with 2·4 per cent. reinforcement the expansion is reduced to a most marked extent.

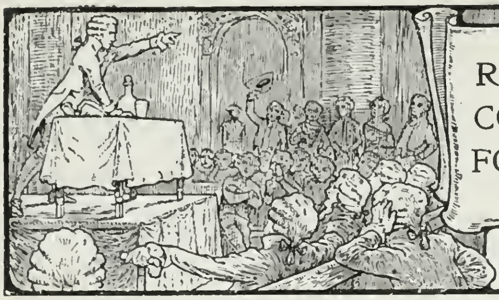
The effect of reinforcement on the expansion and contraction of concrete or mortar, due to changes in its degree of humidity, is therefore in need of further investigation. So far, it appears impossible to derive any general rules relative to this question.

CONTRACTION OF MORTAR EXPOSED TO AIR (SCHÜLE).

—	Unreinforcement.		Reinforcement 1·22%		Reinforcement 2·41%	
Cement.	Z.	V.	Z.	V.	V.	Z.
7 days	0·000096	0·000132	0·000060	0·000084	0·000117	0·000117
28 ,,	0·000498	0·000510	0·000363	0·000318	0·000294	0·000351
84 ,,	0·000660	0·000738	0·000366	0·000366	0·000348	0·000276
210 ,,	0·000642	0·000741	0·000144	0·000420	0·000273	0·000210
1 year	0·000867	0·000996	0·000384	0·000534	0·000432	0·000369
1½ ,,	0·000774	0·000840	0·000333	0·000492	0·000351	0·000351

EXPANSION OF MORTAR IMMERSED IN WATER (SCHÜLE).

—	Unreinforcement.		Reinforcement, 1·22%		Reinforcement, 2·41%	
Cement.	Z.	V.	Z.	V.	Z.	V.
7 days	0·000141	0·000072	0·000081	0·000042	0·000033	−0·000045
28 ,,	0·000261	0·000132	0·000144	0·000171	0·000150	−0·000015
84 ,,	0·000306	0·000168	0·000231	0·000174	0·000135	+0·000066
210 ,,	0·000345	0·000240	0·000294	0·000201	0·000180	0·000030
1 year	0·000330	0·000249	0·000267	0·000186	0·000150	0·000051
1½ ,,	0·000342	0·000282	0·000249	0·000237	0·000150	0·000072



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

SOME PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

INSTITUTE OF CHEMISTRY.

CEMENT.

By BERTRAM BLOUNT, F.I.C.

Mr. Bertram Blount delivered his second lecture on "Cement" at King's College on Friday, December 1st, Professor Raphael Meldola, F.R.S., Vice-President, occupying the chair.

PROCEEDING to the more practical details of his subject, Mr. Blount pointed out that, owing to our limited knowledge of the chemistry of cement, it is necessary to resort to tests of a physical and mechanical nature in order that the user may be able to satisfy himself that the product he employs will meet the demands laid upon it. From their very nature, such tests must be arbitrary, and, to ensure uniformity, they must be carried out under standard conditions which will satisfy the requirements of the consumer and not lay too heavy a burden on the producer. Mr. Blount was assisted by Mr. Gillett, who demonstrated the method of mixing the cement to be examined, on which so much of the reliability of the tests depends—a point which was illustrated by the story of a firm who undertook to subject a cement to mechanical tests and reported that a briquette had been made and had been found to contain a large number of voids. Since cement is not deliberately used under tension, it would appear at first sight to be more logical to appraise its mechanical strength by compressive rather than tensile tests; but, as all the tests are merely used as a basis for comparison, and the errors inherent in compression tests largely invalidate the results, it has become customary to omit them in favour of tensile tests of either the neat cement or of mixtures of cement and sand of a given degree of fineness. Much prominence has been given to the advantage of testing mixtures of cement and sand on the ground that such a method approximates more closely to the conditions obtaining in actual practice; whilst, on the other hand, it can equally be urged that the sand prescribed by any standard specification will differ widely both in size and shape from the aggregate usually employed on a commercial scale.

For the user of cement the question of soundness—*i.e.*, the property of remaining constant and unalterable in value—must always be of paramount importance, and justifies the decision of the British Standards Committee in adhering to the simple Le Chatelier test, which has not infrequently been subjected to severe and rather adverse criticisms in some countries.

The methods prescribed in the Standard specifications of to-day are virtually the same as those laid down some seven years ago, and the quality and uniformity of the modern product have been so well maintained that almost the only alterations have been in the direction of increased stringency, without thereby laying an unreasonable task upon the producer. The information obtained from the ordinary physical and mechanical tests is sufficient for all practical purposes, and probably will not be usefully augmented until some methods are devised for determining the proximate composition of cement and the setting time of concrete. That the former of these has not yet been realised is shown by the fact that, in spite of numerous attempts, no reliable method exists of ascertaining the content of so-called "free lime" in cement.

Turning to the economic side of the subject, it was pointed out that, mainly through the introduction of the rotary kiln, the cost of production of Portland cement had been so far reduced as to render possible the application of the material to uses in which lime had previously held the field; so that at the present time a limit can hardly be set to its sphere of usefulness. It was shown that, whilst theoretically concrete should weigh about 162 lb. per cu. ft., the figure found in practice rarely exceeds 140 lb.; or, in other words, about 14 per cent. consists of voids—a fact not to be ignored by engineers. This consideration at once suggests the possibility of increased economy in construction by using every endeavour to make concrete denser and consequently more efficient. Not only may a saving of material be effected thereby, but a further advantage is gained from the fact that the denser product will be less pervious, and better able to resist the action of water. It must be borne in mind that water continually exerts a solvent action on cement, and it is obvious that the life of the concrete depends in a large degree on its power of resisting penetration.

For structures of concrete exposed to the action of water the liberal use of cement is an obvious means of securing imperviousness; but the employment of some substances—*e.g.*, of the puzzolanic class—which will combine with the hydrated lime set free in the process of setting of cement is perhaps not so common. In the selection of a suitable aggregate not only must its texture be taken into consideration, but also the possibility of any chemical action that it may exert in the concrete; thus debris containing sulphates or easily decomposable silicates is unsuitable. So much has been said of the danger of "magnesia in cement" that the amount allowed has been reduced to quite a low figure. Undoubtedly there has been considerable confusion over this point; the presence of magnesium compounds in structures that have failed under sea-water conditions was really due not to the presence of magnesium compounds in the original cement, but to the formation of magnesium hydroxide by the reaction of the magnesium salts in sea water with the lime compounds of the cement.

After thus outlining a few of the intricate problems that may be presented to the technical adviser, Mr. Blount sketched out a course of training for those who intended to specialise in the subject. After a broad elementary education, he insisted on a sound general knowledge of chemistry, supplemented, if possible, by some mechanical and engineering training, before attempting to proceed to a post-graduate training in the chemistry of cement. This should be gained in a laboratory within easy reach of some centre of the industry where access to works can be obtained, in order that the student may be able to free himself from the small details of laboratory practice and grasp the broad principles, technological and economic, that underlie success in any chemical manufacture.

In conclusion, Mr. Blount briefly reviewed the present position of our knowledge of the chemistry of cement, and paid a high tribute to the valuable results achieved many years ago by Le Chatelier, which remain but slightly modified to-day.

NOTTINGHAM ARCHITECTURAL SOCIETY.

REINFORCED CONCRETE.

Paper by MR. HARRY BURKE.

At their meeting on Tuesday, November 21st, the Architectural Society of Nottingham invited the Nottingham and district master builders to hear a Paper by Mr. Harry Burke, C.E., of Manchester, on "Reinforced Concrete." The President, Mr. Evans, F.R.I.B.A., occupied the chair, and there was a large attendance.

Mr. Burke, who has had considerable experience in the design and erection of work generally in reinforced concrete, directed his remarks at the outset to the comparison between the architect's and the consulting engineer's position and that of the engineer for the various reinforced concrete systems, stating that the time had come when both the engineer and the architect should be in a position to examine in detail any design sent and become master over the work whilst in progress, which would ensure a thorough understanding and give him confidence.

The lecturer then passed on to the advantages and disadvantages of the construction, and particularly dealt with reinforced concrete against steel frame construction. He instanced three examples to show in ordinary architectural work where steel frames would be more advantageous; and, on the other hand, where reinforced concrete would be more suitable than either steel frames or ordinary construction.

Mr. Burke agreed that in an engineering works the "bay system" (which modern practice shows is the test for economy and ensures convenience in the handling of material) was most suitable. To construct such works in reinforced concrete would not be so suitable as the steel frame, for the reason that in the latter case the stanchions can be easily arranged to carry the crane and roof girders which support the roof principals. Again, the cost would be less than reinforced concrete, and the building much lighter both in appearance and structure. The time of erecting also would only be a portion of that taken with reinforced concrete.

He next dealt with house and villa work, showing that, as concrete has a cold nature, it would not be so satisfactory nor so cheap as brickwork, while, in the summer, there would be the chance of condensation and sweating of the walls. Mr. Burke then turned his attention to warehouse and large factory premises, where a building must be as fire-resisting as possible, and illustrated by diagrams that such a structure is far superior and cheaper in reinforced concrete.

After dealing with the theory of reinforced concrete, pointing out how to obtain the neutral axis, moment of resistance, and the percentage of steel for various crushing stresses of concrete, models were shown illustrating a column and beam built up in reinforced concrete, forming part of a perfect monolithic structure which is practically impossible in any other form of construction. Great stress was laid on the method of stirrupping reinforcement so as to ensure all the bars being placed accurately in position before any concrete is put in. This gives facility for a thorough inspection and satisfaction on this vital point.

Mr. Burke also spoke on the respective merits of the bond and plain bar, showing the advantages of a bonded bar, and explained that the cost, if designed thoroughly, was no more than for plain reinforcement, while the resistance was greater. He then dealt with rusting action on the reinforcement, the chemical action on a reinforced concrete structure, and also on the electrical influence which could act on such a structure; and afterwards dealt with the various aggregates suitable for reinforced concrete.

The lecturer went very closely into the costs of reinforced concrete work which he had recently carried out. The first example was the reconstruction of a two-storey galvanising works which had to be as fire- and acid-resisting as possible. The works are 200 ft. long by 180 ft. wide, spaced into bays of 20 ft. by 18 ft. The height is 16 ft. 6 in. from ground floor to first floor, and from first floor to roof 14 ft. The loads to be carried were 2½ cwt. per sq. ft. for first floor, with certain positions on this floor to carry acid baths of 20-ton weight, and hoods 40 ft. by 7 ft. had to be slung from the ceiling. The roof was designed for 1¼ cwt., allowing in each case a factor of 4. The roof is a flat one, and a portion is kept level without fall for building pattern stores, etc. The superficial area is 8,000 yds. (two floors). The contract schedule price was £11,904. In this amount the following items were allowed for which do not form part of the building contract:—

	£
Cast-iron plates round baths	500
Fans for ventilating	150
Hoods for fumes	320
	£970

Leaving the contract in round figures at £11,000, which is equal to 27s. 6d. per superficial yard, or practically twopence per cubic foot. In addition to the amounts included in the £11,000, £500 was provided for terra-cotta work and £150 for treating the front concrete wall, half of which could be deducted if it was required to be cheapened.

In these works the most interesting part was the comparison of the three schemes so as to get the lowest price from open tender and to compare each method of construction against the other. In the first case, a scheme was prepared on the steel-frame system, afterwards casing the whole in with concrete. Not being satisfied with this, a design was prepared making the steelwork of light sections, such as angles, tees, built up at the maker's yard so as to be easily fixed at the site, and in this scheme full value was taken of the concrete for the compressional members. Lastly, it was designed in reinforced concrete, and the costs from actual tenders were :

	£
Reinforced concrete building	11,994
Light steel frame	13,210
Steel frame cased in concrete	14,200

This clearly shows the saving by using reinforced concrete where the building is of average size and the sheeting can be turned over.

Another interesting example was a floor built over wire ovens and prepared for taking five storeys. The load was 2½ cwt. per sq. ft. Including foundations, superstructure, 4-in. reinforced-concrete walls, cement-finished, and all steel casements and fittings, the cost was 25s. per super. yd. of floor covered, two floors only being erected.

Another piece of work recently executed was a floor with cantilever 10 ft. 6 in. long, built over ovens again, but in an existing building. The area covered was 800 yds., and the price when carried out for foundations 3 ft. below the ground was 19s. 6d. per super. yard.

For foundation work, Mr. Burke gave a very interesting example he was consulted upon in Liverpool eighteen months ago. A site had been chosen for a public building, and after the contractors commenced excavating they found that half the building would come upon a quarry partly worked, and half upon that which had been worked and had been filled up 90 ft. The design eventually carried out was the sinking of two concrete piers 90 ft. to solid earth, and from these reinforced concrete beams, 7 ft. 6 in. deep by 24 in. wide, were carried and cantilevered so as to take the new building. The cost was about £1,000, but what would the cost have been to go down to the solid to prevent chance settlement and the building breaking its back?

Another interesting example was a scheme of precipitation and sludge tanks for treating 1,000,000 gallons daily of works sewage. Space was limited and a scheme of tanks was prepared 250 ft. 12 in. wide with cast-iron plates for the sides and bottom. The cost of this worked at nearly £7,000. A design in brickwork in cement was then made, but the capacity could not be obtained without buying more land. Apart from the land, the price worked out to just over £6,000. It was then decided to use reinforced concrete, having internal walls without ties 9 in. thick at the bottom, 6 in. wide at the top and 8 ft. high, and this cost just over £4,500. This set of tanks was, in Mr. Burke's opinion, the first erected in this country in reinforced concrete to give this capacity.

A very interesting example of pile and roadway work, designed for the English Velvet and Cord Dyers' Association, at Greenfield, was shown. They wished to enlarge their works, but they were handicapped, having the river on one side, a goyt (for supplying the works with water) on the other side, and beyond this a roadway and canal. On the one side the river was dammed and a reinforced concrete raft made, from this ran the columns and beams for forming a new roadway over the river, and at the same time carry the buildings to be erected, and on the other side 12-in. reinforced concrete piles were driven to carry the new building. The super. yardage of the work done was 1,250, and the total cost was £2,800, excluding the paving. The roadway was designed to carry a 10-ton motor-wagon.

Another piece of work and tenders received for was a new roadway 36 ft. wide to carry two traction engines passing side by side. The suspended area carried by columns, beams and slabs was 610 yds, and an equal portion had retaining walls to support the made-up road. The cost of this work was £2,000.

In railway-bridge work an example was shown for Ambergate to carry one set of rails. This has one central pier in the river, the width of which is about 120 ft. There are two 60-ft. spans, and the width of the bridge was 12 ft. The cost from tenders sent in was £850.

Concrete Surface Treatment.—Mr. Burke then dealt with the treatment of reinforced concrete for architectural effect. This must appeal to everyone as very interesting and worthy of serious consideration, in view of the fact that as time goes on this particular method of construction must come more and more into favour. His own experience had been in buildings of the warehouse and heavy works type where the first consideration was utility with very little scope for elaborate design on the elevation, but at the same time the question had been considered, and the following were his views on the subject: Taking the ordinary works or warehouse building with the orthodox piers, plinth courses, stone heads and sills, stone bands with cornices parapet, etc., this type of building in reinforced concrete can be very ably and economically treated as follows. The concrete columns on the outside walls would be designed as piers of the necessary proportions to suit the design, and the frame-beams between them utilised as window-heads, forming a continuous band. Between the framework a light 4-in. reinforced-concrete wall could be constructed connected with the columns and the frame beams; the window openings and sills would, of course, be framed in the sheeting. In case of any mouldings, etc., being required the jambs would be left rough and the moulded work formed at another operation. Rusticated or V if required in the blocking out of the piers can be done by ribs of timber being fastened on to the sheeting before same is fixed, care being taken to prepare the timber with a good wash, and also that these strips are slightly bevelled so as to ensure clean striking. The ballast for this class of work should be very fine and of granite if possible. If the elevations are carefully rubbed to ensure all voids being closed after the sheeting is struck a very good job is assured, and the difference from stone is hardly noticeable. If a combination of brick with stone dressings is required, the framework may be carried out exactly the same way, but the 4-in. reinforced concrete wall would be substituted by a 9-in. brick one, and the introduction of moulded work or terra-cotta cornices is a simple matter.

To review the treatment of a classic building in reinforced concrete, consider the first method. The building of the main walls would be a very simple matter, as any details in the way of cornices, blocking out, and the like, as well as panelling, could be easily executed in the sheeting. With regard to a Corinthian capital, to carry out the shaft alone satisfactorily would be a very difficult job, and, as far as the capital is concerned, would be practically impossible. Then, considering the method of casting independent blocks, which would be placed in position in the same way as is now adopted with terra cotta and reinforced concrete, the question of carving arises. Taking it for granted that such a front could not be executed in concrete alone, the introduction of stone, for reasons stated, becomes necessary. Considering the combination of concrete and stone blocks, the two substances would have to be matched both in texture and in colour to be at all satisfactory. This, of course, could be done with care, but, supposing that a suitable appearance of the work on completion be attained, we must not lose sight of the probable change that would take place through the exposure to weather and general atmospheric conditions. From this the architect would then have to decide whether it would be worth while taking the risk (which would certainly exist) of adopting reinforced concrete with the combination of stone to save the extra cost which an entire stone front would incur.

In conclusion, Mr. Burke illustrated by lantern many examples of work carried out, which proved very interesting to those present.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course o, construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE UNDERGROUND STORAGE TANKS, HANDSWORTH.

The underground storage tanks at the Witton Sewage Pumping Station were constructed in reinforced concrete on the Hennebique system. These tanks are made in duplicate. They have a total capacity up to the inlet level of 264,342 gallons, and they vary in depth from 19 ft. to 20 ft., and have been calculated to withstand a pressure of water from without, when empty, equal to this head. To obviate the danger of the tanks being bodily lifted, when empty, by the pressure of the water standing in the ground, the bottom is carried out for a distance of 18 in. all round. The roof is calculated for a superload of 4 cwt. per sq. ft.

The tanks were designed by Mr. H. Richardson, A.M.Inst.C.E., surveyor to the Handsworth Urban District Council, the contractors being Messrs. Joseph Howe and Co., of West Hartlepool.

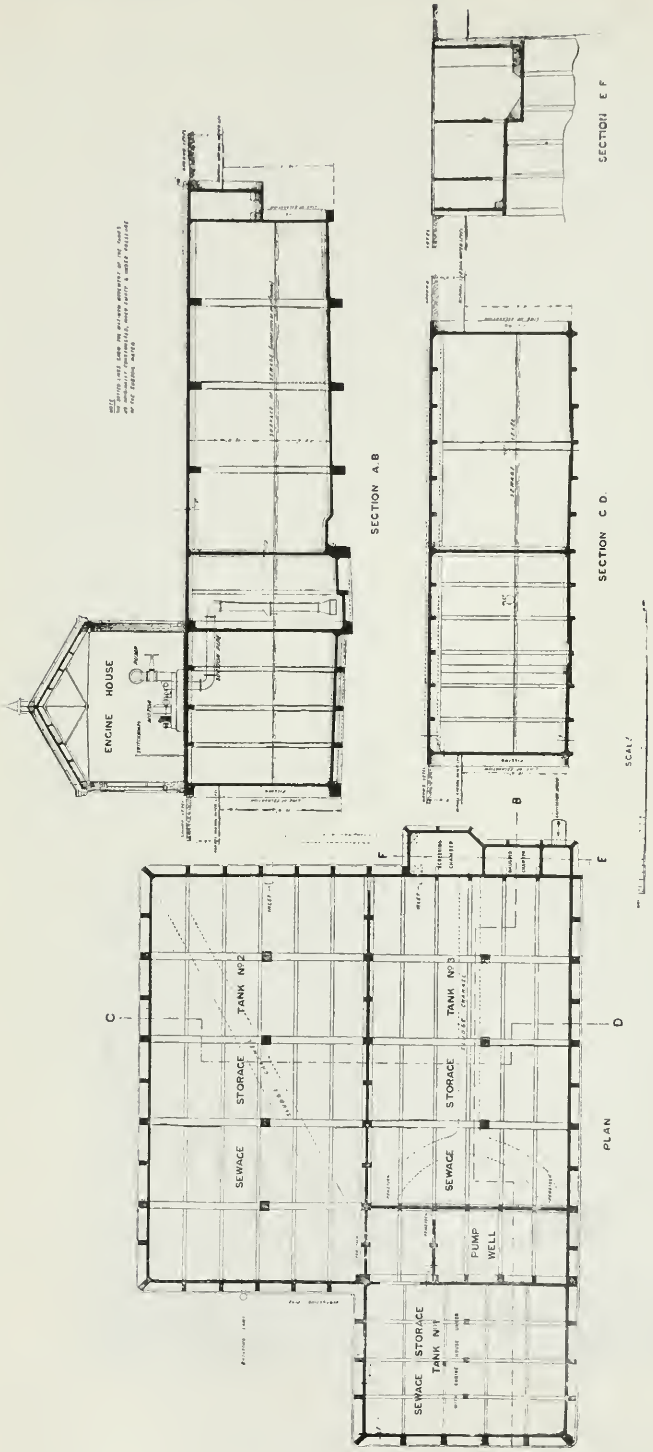
In the course of a paper on "Handsworth," delivered before the Institution of Municipal and County Engineers by Mr. H. Richardson, the lecturer made the following remarks :—

"Perhaps one of the most interesting features from the engineer's point of view in connection with the reinforced-concrete tanks was their behaviour when they were subjected to the pressure of the subsoil water.

"The excavations for the tanks were sunk in very wet soft sand-stone, the site being only 150 yards from the river, and on nearly the same level. The subsoil, 5 ft. down from the roof of the tanks, was principally made-ground and clear of water, but below that level to the bottom of the tanks occurs a soft sand rock which is full of water. This was, of course, drained to a sump, and pumped away during the whole

Inside view of a finished tank.

Reinforced Concrete Underground Storage Tanks, Handsworth.



Plan and Sections.
Reinforced Concrete Underground Storage Tanks, Handsworth.

time the tanks were in course of construction, and when the work was complete, the shuttering was withdrawn, and the working space all round the tanks filled in, the subsoil water pumps were stopped and the water outside the tanks allowed to resume its normal level.

"It was known that the dead weight of the tanks themselves, except a small portion upon which the engine house is built, would be insufficient to maintain their stability against the upward pressure of the subsoil water, and the designers of the tanks adopted means for anchoring them down. The floor of the tanks is extended outwards to make a projecting sill all round 18 inches beyond the outer face of the walls. This should have had the effect of anchoring down the tanks and enabling them to resist the upward pressure of the subsoil water which stands at a height of 16 feet above the floor of the tanks. As the subsoil water began to attain its normal level it was found to be lifting the tanks at the end farthest from the engine house, and as the water increased in height so the tanks came farther out of the ground. The maximum displacement was 6½ inches. The subsoil water pumps were at once re-started and the water level lowered, water being at the same time admitted into the tanks so that the levels inside and outside were soon equal, the result being that the tanks went back into position with practically no damage done to the tanks and none whatever to the engine house, the only serious fracture being where the inlet pipes passed through the walls of the tanks. Apparently the reason why the anchor or sill was ineffective was because it was covered only with the filled-in material around the tanks, which compressed as they rose. This was afterwards remedied by filling the trench all round the tanks with concrete.

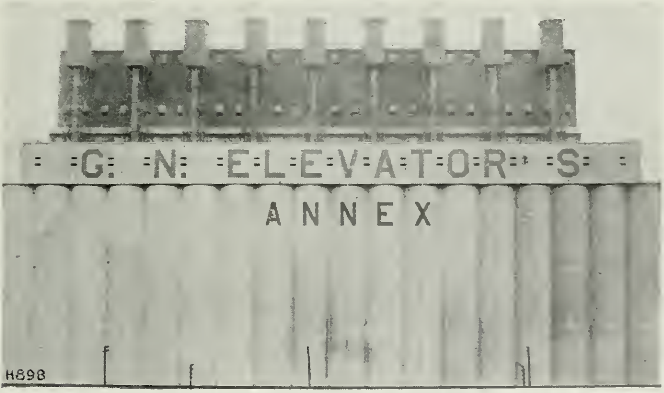
"This is rather an interesting point to engineers, because constructions of this character in brick or ordinary concrete, the walls and floors of which would probably average 5 to 6 ft. thick and would generally be heavier than the water which they displace, would require no special means to keep them in position when once constructed, but with reinforced concrete construction, which is comparatively of such a light nature, special means must be provided for anchoring them down in water-logged ground, or they would be liable to float like a ship.

"The strains were severe and complex to which these tanks were subjected by the pressure of water when being held down by the weight of the engine-house over a very small area at one end, and also by the large cast-iron pipes which were built into the wall at the other end.

"In the author's opinion, it is a tribute to the value of reinforced-concrete construction that, in spite of the extraordinary strains to which it was subjected, the whole of the work settled back into position as a monolithic construction with practically no damage done."

REINFORCED CONCRETE GRAIN BINS FOR THE G.N. RLY. AT SUPERIOR, WIS., U.S.A.

The reinforced concrete grain bins described in this article were erected to replace a large wooden structure previously used for storage purposes, and which was destroyed by fire in 1908. It was decided to make this new storage-house an extension of the large steel working-house, erected in 1909.

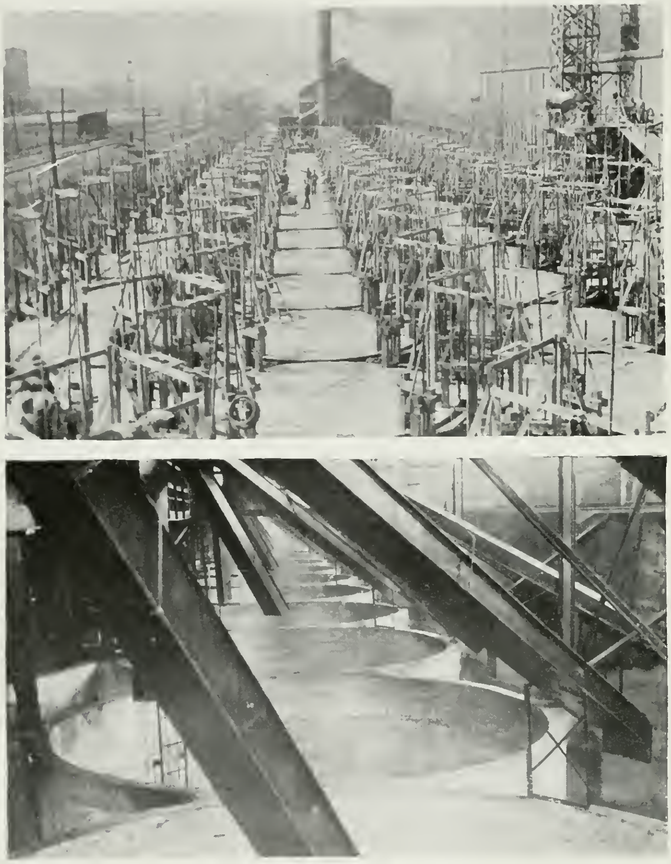


Reinforced Concrete Grain Bins, Superior, U.S.A.

The new bins have a storage capacity of 2,400,000 bushels.

As the height from the main floor to the bin floor of the steel house is 117 ft., this required unusually tall storage bins. A height of 110 ft. for the bins was fixed upon.

Owing to the nature of the soil at the site of the storage-house, a complete pile support was decided upon. The height of the grain-bins is so great, and the resulting load on the foundations so heavy, that it was necessary to drive piles over the entire area of the excavation. Approximately, 5,000 of these piles were driven. Practically the whole of the load on the piles, amounting to $3\frac{1}{2}$ tons per foot over the entire area, is delivered to the bottom slab, and thus to the piles, through the outside walls and subdividing walls of the basement. Except at the belt tunnels, these walls were placed so close together that there was no necessity for designing the bottom slab as a beam. The footings of the walls, with a projection of 1 ft. each side of the wall, together with the 2-ft. depth of the slab, were sufficient to care for the distribution of loads to the piles. At the conveyor tunnels, however, it was necessary to have a 7-ft. clear space between walls, and, in order to get the proper amount of load to the piles under the middle of the tunnels, the upper part of the slab was reinforced with 1-in. sq. corrugated bars placed 4 in. centre to centre. Three-fourths of these bars were turned down to take shear and diagonal tension. In order to tie the whole of the slab together, the fourth bar was made continuous for the full length of the basement, using 27-in. laps at points of no stress.



Grain Bins, Superior, U.S.A.

The second problem in distribution was to get the loads from the bin walls into the basement walls, and from these into the slab. With bins of the height and diameter used, nearly the whole of the grain load is carried by the walls, and this, added to dead loads, gave a total of about 45,000 lb. per lin. ft. of wall at the bottom of the bins. In order to distribute this load, 20-in. standard I-beams, weighing 80 lb. per ft., were used over the conveyor tunnels. Reinforced concrete girders 3 ft. deep and 8 ft. 9 in. long, were used for the shorter spans between subdividing walls. As it was necessary that the slab and walls should be built at different times, inclined bars were built into both to take care of the longitudinal shear.

Grain pressure on the bottoms of the bins was taken care of by a reinforced concrete slab 6 in. thick, spanning the short spaces between subdividing walls of the basement.

In the construction of the basement slab and walls 1 : 2½ : 5 concrete was used, both sandstone and gravel being used as aggregate. Medium steel corrugated bars were used throughout.

In designing the bin walls, the thickness at the bottom was determined by the compression due to vertical loads, except that the walls were made somewhat thicker than theoretically required. The outer walls, having a grain friction on one side only, were made 7 in. thick—½ in. thinner than the inside walls. The extra ½ in. of thickness of the inner walls was considered advisable, not only on account of the greater vertical pressure, but because of the fact that these walls act as arches when circular bins are empty and the adjacent interspace bins are full.

Bursting pressure in the bins was cared for by using plain round bars, ⅝ in. and ½ in. in diameter, spaced according to the bursting load. The circular bins were tied together, and the bursting pressures of the interspace bins provided for by corrugated bars bent into U-shape, placed in the intersections of the bins.

All concrete placed in the bin walls consisted of one part sand to two parts gravel, with 1½ barrels of cement per cubic yard of finished concrete. As both sand and gravel were well graduated in size, these proportions produced an exceedingly rich concrete.

The cupola consists of a steel framework, resting on the bin walls enclosed by and supporting 6-in. reinforced concrete side-walls. These walls were reinforced for wind pressure only. The roof consisted of reinforced concrete tile, resting on steel tees, and covered with 4-ply pitch and gravel roofing. The roof tiles are 18 in. span, 24 in. long, and 1⅛ in. thick, reinforced with woven wire.

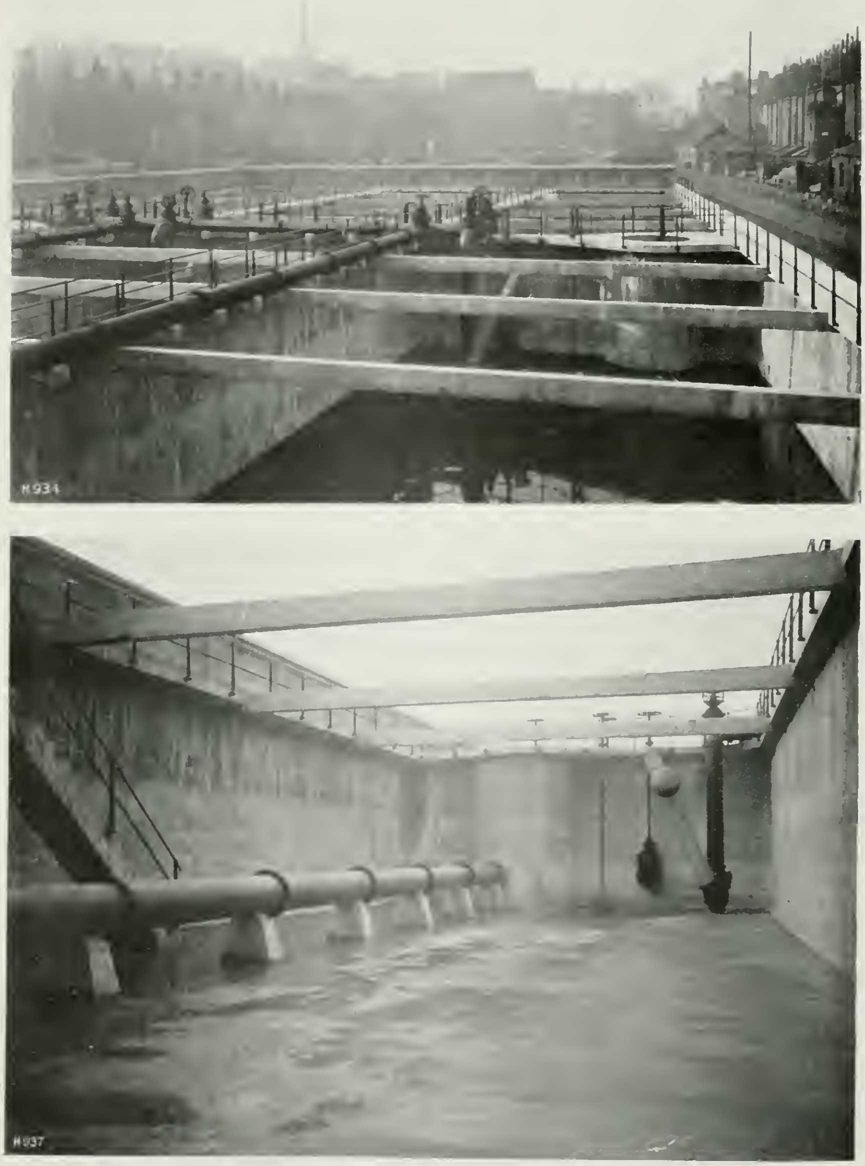
After the completion of the concrete work, all exterior concrete surfaces were treated with a cement and alum wash, both as a partial waterproofing and as a finish.

The work was designed and constructed under the direction of Mr. A. H. Hogeland, Chief Engineer, Great Northern Railway. Mr. O. B. Robbins, from whose article in *Engineering News* the above particulars are taken, was the engineer in charge of all construction work except excavation and pile-driving.

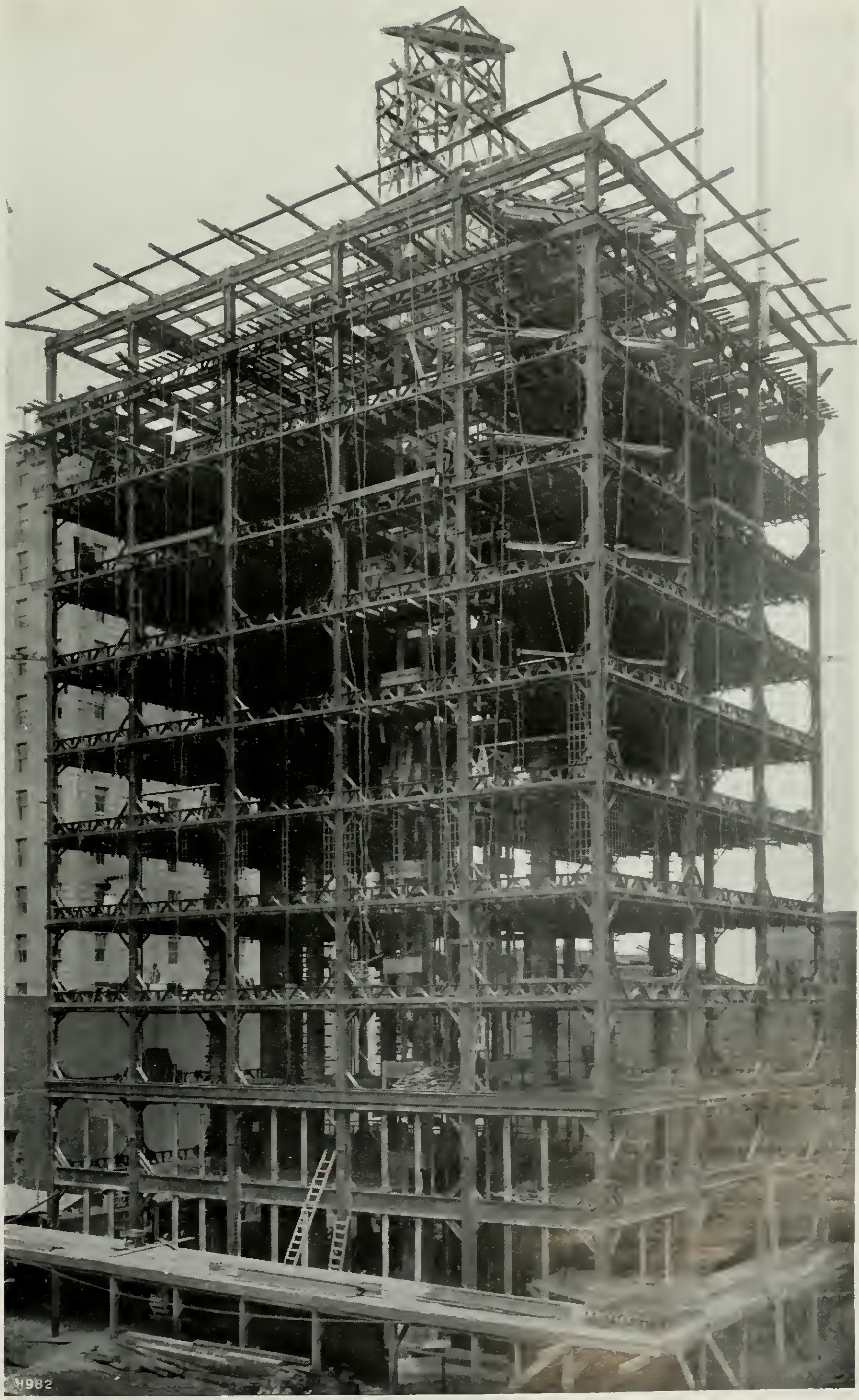
Pumping Station for the London Hydraulic Power Co.

REINFORCED CONCRETE IN THE NEW PUMPING STATION FOR THE LONDON HYDRAULIC POWER COMPANY.

Reinforced concrete was largely used in the construction of the new pumping station for the supply of hydraulic power in London which was opened recently at Grosvenor Road, Westminster. The station, which belongs to the London Hydraulic Power Company, comprises reservoirs for water settling, an engine- and boiler-house, hydraulic



Pumping Station for the London Hydraulic Power Co.



Building in course of construction.
"EARTHQUAKE-PROOF" BUILDING FOR THE ROYAL INSURANCE CO., SAN FRANCISCO.

accumulator tower, coal store, filter-house, overhead tanks, etc. The floors throughout the building and in the coal stores are of reinforced concrete on the Hennebique system. The reservoir and tanks are also constructed of this material.

The reservoir is divided into six compartments, each about 100 ft. long, 38 ft. wide, and having a depth of 16 ft. from top of gangways, with a total holding capacity of about $2\frac{1}{4}$ million gallons. A view of the reservoir in course of construction is shown on page 66, as well as an interior view of one of the tanks when empty. The water is conducted from the river into the different compartments of the reservoir through two lines of 20-in. diameter pipes, laid at half tide level along the subway pipe trench. There are also two lines of emptying or wash-out pipes, 14 in. diameter, laid below the bottom of the reservoir.

The whole of the reinforced concrete was done by Messrs. Mouchel and Partners, of Victoria Street, and the contractors for this work were Messrs. John Mowlem and Co., of Grosvenor Wharf.

"EARTHQUAKE-PROOF" BUILDING FOR THE ROYAL INSURANCE CO., SAN FRANCISCO.

The building described in this article, and which was recently dealt with in *Engineering Record,* was built to replace the premises of the Royal Insurance Company in San Francisco which were demolished by the earthquake of 1907. The design of the new building was specially studied by the architects and engineers with a view to earthquake shocks.

The study comprised special bracing, designed to resist transverse stresses and vibration, special methods of thoroughly anchoring masonry and terra cotta to the framework, and especially a system of vertical and horizontal tension rods, forming a sort of large mesh around which the brick walls are built and are thoroughly bonded through and through it, with a result which can be compared somewhat to the reinforcement of wired glass, in that, even if the wall were badly cracked and shattered, it is not likely that any portions of the wall would fall to the street.

The solid floors form continuous horizontal diaphragms, and the basement floor has great mass and strength for the purpose of offering high lateral resistance and giving additional security to the bases of the columns. All of the structural steelwork is fireproofed, and the use of wood has been entirely eliminated.

The columns, of ordinary plate and angle construction, are spaced from 16×16 ft. to $16 \times 17\frac{1}{2}$ ft. apart, and have cast-iron bases seated on I-beam grillages with concrete footings on the sand about 18 to 24 ft. below the surface of the ground and 14 ft. below ground water-line. Each column is anchored by four large vertical bolts fastened to its base and through the grillage, and is enclosed in concrete or brickwork. The grillages are enclosed in solid concrete up to the basement floor, and the space between them is covered by the floor of concrete, 30 in. thick, reinforced by two crossed layers of steel rods $1\frac{1}{8}$ in. in diameter and about 30 in. apart on centres. At the columns all rods are bent 270 deg. around the columns and continue at right angles to the former direction to the next column, where they are bent 270 deg. and are continued again in another right angle, and so on, six or eight rods being thus bent around each column and forming a basket-like network to afford tensile strength in every direction.

The wall columns are connected by lattice girders at each floor, with knee-brace angles at both ends and on both flanges, thus forming stiff belts around the building at every storey.

All floors are of concrete with electrically-welded reinforcement and cement finish. All partitions are of cement mortar on wire lath and metal furring. The traction type electric elevators are enclosed in fireproof steel frame shafts with the operating machinery on top.

Messrs. Howells and Stokes, of New York, were the architects of the building. The Thompson-Starrett Company, of San Francisco, were the general contractors, and the structural steelwork was made by the American Bridge Company.

NEW USES FOR CONCRETE AT HOME AND ABROAD.

Under this heading reliable information will be presented as to new uses to which concrete and reinforced concrete are put, with data as to experience obtained during the experimental stage of such new applications of these materials. The use of reinforced concrete as a substitute for timber in exposed positions is one of the questions of the moment. Railway sleepers, telegraph posts, fence posts, etc., of concrete are being tried. Similarly, efforts are at present being made to prove that reinforced concrete is an excellent substitute for brickwork where structures of great height are required.—ED.

REINFORCED CONCRETE PRESSURE PIPES, UMATILLA, U.S.A.

THESE pipes were required to distribute the water in the Umatilla project of the United States Reclamation Service.

Reinforced concrete was found to be the best material to use, and in the season 1906-7, 4,832 lin. ft. of 46- and 47-in. concrete pipe were made and 3,868 lin. ft. of 30-in. pipe. Considering one sack of cement as 0·9 cu. ft., the proportions by volume were approximately 1 part cement, 2·3 parts sand, and 3 parts gravel. The reinforcement used consisted of $\frac{5}{16}$-in. mild steel wire wound on a drum into a helical coil. The spacing was such as to permit a maximum stress of about 12,000 lb. per sq. in. A view of the forms is seen in *Fig.* 1. The main forms consist of an inner core of $\frac{1}{4}$-in. steel plate 8 ft. in length, with a key-piece. The inner core is hinged to permit its being collapsed when the key-piece is removed. The collars were of concrete, the same mixture being used as for the pipe. They were 3 in. wide and 3 in. thick, reinforced, where the head was over 50 ft., with No. 3 rib stud supplied by the Trussed Concrete Steel Co.

The experience with the pipe lines laid during the previous year was sufficiently gratifying to warrant the making of concrete pipe during the season 1908; 7,056 lin. ft. of 46-in. concrete pipe were made and 9,216 lin. ft. of 30-in. pipe.

In 1909, owing to the satisfactory experience of the two previous years, a further length of 9,800 46-in. concrete pipe was laid. This would have to act under a maximum head of 110 ft. The work of laying this began in November, 1909, and the line was finished in January, 1910. The pipe was tested in February, 1910, it being filled in two hours and forty minutes. There were no mishaps, and no leakage was apparent.

During the season of 1910 a further 4,000 lin ft. of 30-in. pipe were made, and

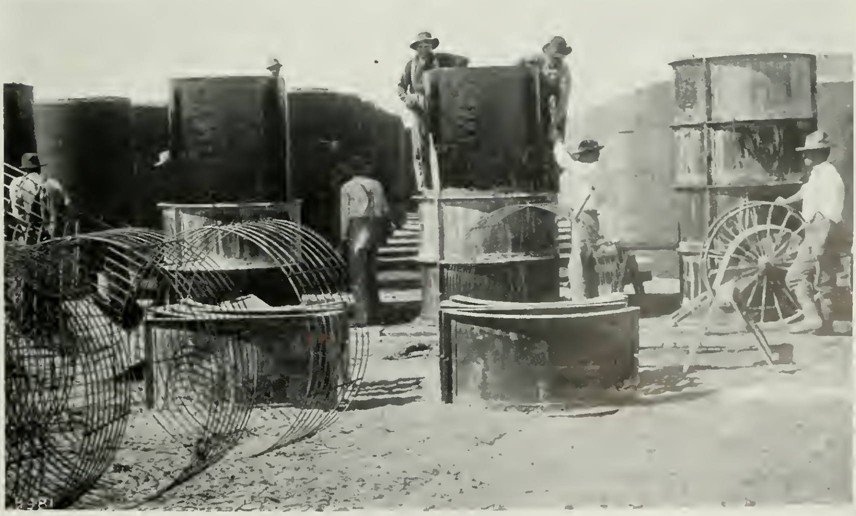


Fig. 1. Steel Forms and Reinforcement.

REINFORCED CONCRETE PRESSURE PIPE, UMATILLA, U.S.A.

the experience with the whole of the reinforced concrete pipe on this project has been thoroughly satisfactory.

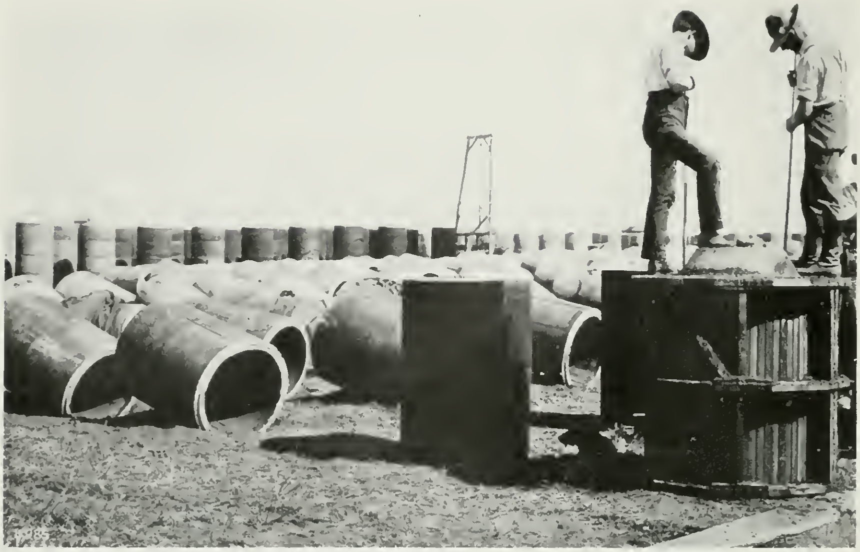


Fig. 2. Some of the Finished Pipes and the Wooden Forms for 30-in. Pipe.

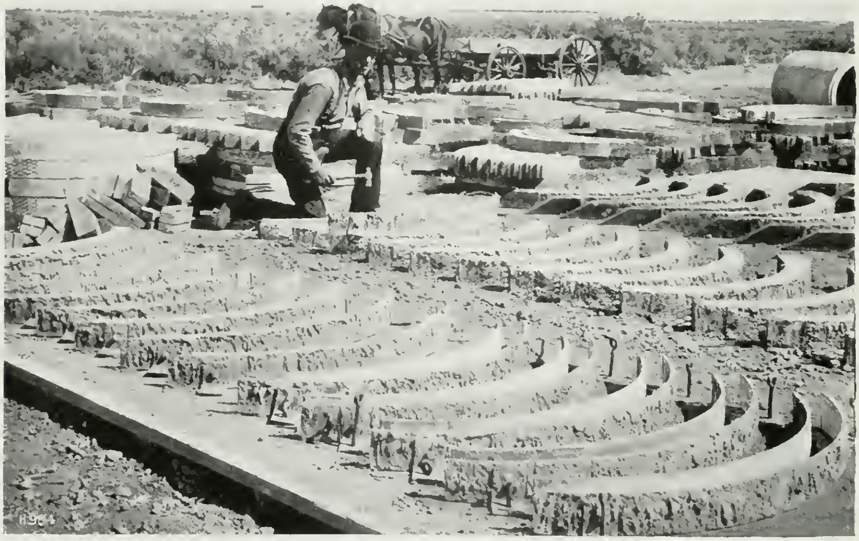


Fig. 3. Forms for Pipe Collars for Reinforced Concrete Pipe.

Reinforced Concrete Pressure Pipe, Umatilla, U.S.A.

Our particulars are taken from an article in *Engineering News*, by Mr. Herbert D. Newell, Project Engineer of the United States Reclamation Service, to whom we are indebted for our illustrations.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months

"Reinforced Concrete Compression Member Diagram." By Chas. F. Marsh, *M.Inst.C.E., M.Am.Soc.E., M.Inst.M.E.*

Published by Archibald Constable & Co., Ltd. Price 3/6 net.

The rules for calculating hooped reinforced concrete pillars, which are given in the Royal Institute of British Architects' Joint Committees' Second Report and in the new London County Council Regulations, are well drawn on a logical basis, and result in formulæ simple in form, though somewhat tedious to use just as they are. We may therefore expect to see various attempts at constructing diagrams having as object the speedy determination of such pillars in practice. The first to reach us is one devised by Mr. Charles F. Marsh, the well-known author of various books on reinforced concrete. This diagram consists of four graphs on one sheet all continuously related to each other. The manner of use for designing a pillar is, roughly, as follows: Taking the load as starting point we trace to an intersection with a line for the size of pillar selected, red lines showing circular cores, and blue square cores. From this first intersection we trace on to intersect with another line, representing the proportion of vertical steel, and then on to meet a line representing, either by its red, green or blue colour, the kind of concrete chosen and the value of a constant chosen from a table and dependent upon the kind of binding and its spacing. We thus derive a ratio of volumes of concrete and hooping, and by the aid of another diagram we ascertain a factor which gives the size of the binding, having regard to the previously-chosen spacing. One can, of course, check a pillar already designed by working back the opposite way.

The author deserves to be highly commended for having gone to so much trouble to devise so useful a diagram. All could wish it were easier to design hooped pillars than even this diagram makes it. Perhaps we shall now have someone go one better. A particularly good feature of this diagram is, however, that it clearly marks the limiting values to be observed. Upon the diagram is explanatory letterpress. The only adverse criticisms we have to make are that the diagram gives both the vertical and horizontal steel as ratios, which means that when these are worked out in sizes of rods it is a chance they may not be commercial sizes, so that we might have to make several trial calculations before a practical result were obtained. A small matter, but one that is worth correction, is the author's use of the words "helical spirals," which are contradictory of each other. What is meant is "cylindrical helices."

"The Composition and Strength of Mortars." By W. J. Dibdin, *F.I.C., F.C.S., etc.*

Published by the Royal Institute of British Architects 9 Conduit Street, Regent Street, W. Price 5/-.

In November, 1907, the Science Standing Committee of the Royal Institute of British Architects drew up a scheme of procedure for an extensive series of experiments into the tensile, crushing and adhesive strengths of mortars and their chemical composition, which were entrusted to Mr. W. J. Dibdin, who presented this report in February, 1911. It has just been published. The experiments are valuable, as all such experiments must be. The cements used were white chalk lime, Dorking greystone lime, and blue lias lime from Stockton, near Rugby. The sands used were Leighton Buzzard standard, Charlton fine, ordinary pit, Thames and ground brick, the last made from grey Kent stock, not overburnt, as commonly used in London buildings. The mixtures used were 1 part lime to 2, 3, 4 and 5 parts of sand, and experiments were also made on the effect of additions of 5 per cent. of clay to 1 : 3 mixture. The adhesion tests were on 1 : 3 mixtures with stock and Fletton bricks. The periods of testing were: 1 month, 3 months, 12 months, and 2 years. Tests were also made on ancient mortars.

The results may be briefly summarised by saying that they showed the influence of the proportion of voids in the sands, and that the poorer limes lose strength on drying out. Blue lias lime was very superior to the others. Current practice seems to favour a 1 : 3 mortar, but the tests showed considerable superiority of the 1 : 2 mixtures, and seeing that the good ancient mortars were seldom of weaker proportions, a 1 : 2 mixture should be generally adopted. The chemical examination of ancient mortars clearly showed the absence of the formation of soluble silica, which it has been assumed was formed in the course of time by the action of the lime in sand—a mistake undoubtedly arising from the presence of pozzuolana in ancient mortars. Clay up to 8 per cent. seems to have no deleterious effect, and Fletton bricks were markedly superior to stock bricks as regards adhesion. The report is supplemented by results of authoritative German tests on lime mortars in which trass (*i.e.*, pozzuolana) has been employed, which clearly shows the improvement in tensile and compressive strength obtained thereby. The practice is therefore to use a smaller proportion of lime to the trass and sand combined. The results of these experiments point to the advisability of others on Portland cement mortars of similar proportions with the same materials, and also upon mortars made with Portland cement and the various limes admixed. We have no hesitation in saying that some very valuable results would thus be obtained, and the probability is that current practice would be much altered thereby, for Portland cement is now sold at such a small price that it would be found advantageous to generally employ it for mortar. Where considerable waterproofness was required it would be advisable perhaps to use lime, or some other material might be substituted for the lime, such as clay. This points to the advisability of further tests upon the porosity of mortars of the composition of those referred to in the R.I.B.A. report, and also upon such mortars as we have suggested should be experimented upon.

DESIGN OF STRUCTURES.

"Framed Structures and Girders—Theory and Practice," Vol. I. Stresses, Part I. By Edgar Morburg, *Sc.D., Professor of Civil Engineering, University of Pennsylvania.*

Published by the McGraw-Hill Book Co., 6 Bouverie Street, London, E.C., and 239 West 39th Street, New York. Price 17/- net.

Contents.—General Considerations and Definitions — The Equilibrium of Coplanar Forces—Application of the Laws of Equilibrium to the Determination of Reactions — Shears and Bending Moments in Simple Beams—Application of the Laws of Equilibrium to the Determination of Stresses—Roof Trusses—Stresses in Bridge Trusses with Horizontal Chords—Modern Bridge Trusses and Their Development—Stresses in Bridge Trusses with Sub-Panels—Live-load Stresses by Graphic Methods; Influence Lines—Skew Bridges and Bridges on Curves—Live-loading for Bridges.

This book is the first of three volumes on framed structures and girders and is devoted mainly to the derivation of the fundamental principles of statics, the determination of shears and bending moments in beams, and the analysis of roof and bridge trusses resting on two supports. Though designed to serve primarily as a text-book, it is also useful as a reference book. Every point seems to be dealt with in elaborate detail, and this volume holds out promise that the whole work will be a valuable one worth the study of every structural engineer. The chapters devoted to bridge-construction are particularly full and valuable. We shall be better able to criticise when we see the second and third volumes to which this is preparatory.

"Reinforced Concrete Construction (Elementary Course)." By M. T. Cantell, *Licen.R.I.B.A.*

London: E. & F. V. Spon, Ltd., 57 Haymarket. Price 4/6 net. 136 pp.+vi.

Contents.—Symbols—Advantages of Reinforced concrete—Mixing and Depositing Concrete—Aggregate for Concrete—Cement—Reinforcement — Bending Moments and Shear Stresses—Additional Information and Data for Designing — Beams with Single Reinforcement—Beams with Double Reinforcement—Slabs—Tee Beams—Main Beams for Floors—Shearing Reinforcements—Adhesive Stress — Inclined Stirrups — Columns — Long Columns — Eccentrically Loaded Columns—Formulæ—Tables—Areas.

This is a very cheap volume, and one that should be appreciated by students, containing as it does a great deal of information and a very clear explanation of the elementary principles. There is a great need for a really good book which would enable students to grasp these elementary principles, as far too often we find that after reading a few pages the reader is faced with complicated formulæ only imperfectly explained as regards their construction, and in consequence the learner is apt to abandon the subject until such time as he may meet someone who will instruct him personally, which may possibly mean that such a time will never come. We must realise that the students of to-day must be well looked after and efficiently trained if good work is to be produced in the future, and too much cannot be done to effect this object. This should prove a very useful book, and is well written, and we are pleased to find that the construction of the various formulæ is explained at the end of the volume, as it is a great mistake to give formulæ to students without explaining their derivation and construction, because they will then be used without a proper understanding and become merely a matter of memory. It is rather surprising that the author does not make any mention of machine-mixing, although hand-mixing is fully described, and in view of the importance and superiority of the former for large works this must be considered as a serious omission. The illustrations can hardly be considered satisfactory, as the diagrams are, generally speaking, far too small and not sufficiently explicit, and we should like to have seen this part of the work dealt with in a more masterly manner.

"Reinforced Concrete Construction in Theory and Practice.—An Elementary Manual for Students and others." By Henry Adams, *M.I.C.E., etc.*, **and Ernest R. Matthews,** *A.M.I.C.E., etc.*

London: Longmans, Green & Co., 39 Paternoster Row
Price 10/6 net. 316 pp. + xiv.

Contents.—History of Reinforced Concrete —Uses, Advantages and Properties — General Principles of Stress — Moments of Resistance—Loads and Reinforcement — Notation, Formulæ and Examples—Special Constructions —Effects of Heat and Frost—Reciprocation, Duality—Abridged Notation and Projection—Reinforced Concrete in Railway Engineering—Reinforced Concrete on Wharves, etc.—Reinforced Concrete in Building Construction— General Notes.

The authors of this volume are so well known to our readers that they will need no introduction, and we should naturally expect an excellent production from such a combination of authors. The book is certainly very complete, and every aspect of the subject has been dealt with and illustrated, wherever possible, with practical examples. The diagrams and illustrations generally are clear and good, and this is an important consideration in a work of this kind. We are glad to see that the general principles of stress are dealt with fully, and this portion, together with the chapter dealing with the moments of resistance, loads and reinforcement, should be of great assistance to the elementary student, for whose guidance they were written. The notes on the effects of excessive heat on concrete are very interesting, especially as this material is claimed to be eminently fire-resisting, and the authors' deductions, which were arrived at after a series of tests, are such that they confirm the right to this claim. It is necessary, however, that the concrete should have ample time to set before the heat is applied, and in the ordinary course of events this would naturally be the case. From the notes on the effects of frost on concrete, it is shown that mixing should not be conducted when the temperature is below 29·3° F., and that the mixing of concrete with warm water in frosty weather, which has been recommended by some engineers, is detrimental. A great deal of useful information is given in the general notes, but we should like to have seen more information included with regard to the construction of centering and forms, as this is an important branch of the work and one that requires to be fully described in a volume which should appeal very forcibly to the practical as well as the theoretical man. A great number of the practical examples are merely illustrated without being adequately described, and we feel that they could well have been omitted, and a good chapter included on the work of centering, etc.

It is, however, a good book, and one which should be well received by students and experienced men alike, containing as it does a complete treatise on the subject reinforced concrete construction.

MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The Work of the Gross-Lichterfelde Testing Station during 1910.—The Director's report of the work of the great Prussian Testing Station for the year ending March 31st, 1911, contains many points of interest in respect to cement and concrete. It is worthy of note that the staff of the station now numbers 222, of whom seventy-two are University-trained engineers and chemists.

The tests of reinforced concrete during the year, mostly made at the request of manufacturers, include compression tests on ten columns, about 13 ft. long, for the purpose of comparing a special form of looped transverse reinforcement with spiral reinforcement, the concrete mixture and longitudinal reinforcement being the same. The spirally-wound columns broke under loads 40–60 per cent. greater than those furnished with loops—the principal reason being that the arrangement of the latter was such that it was impossible to ram the concrete, which was only distributed by means of a lath. The spirally-reinforced columns were rammed lightly at intervals of 4 to 5 in. Similar results were obtained in testing a series of eight columns 6 ft. 6 in. × 12 in. × 12 in., each with a different form of reinforcement. Spiral and hooped transverse reinforcements gave the highest results.

Four concrete beams, prepared with a 1 : 4 cement-ballast mixture with 7·4 per cent. of water, gave a bending strength of 443 lb. per sq. in. Many experiments were also made to determine the resistance of reinforcing rods to slipping in concrete of various compositions, and the permeability of other mixtures to water.

The officials are often called upon to determine whether the concrete in some building was originally of the right composition or not. The only methods at present available for this purpose involve the chemical treatment of the specimen, and it is pointed out that these methods can only be applied when the sand and aggregate are free from substances soluble in acid. This is, of course, the case when quartz sand and gravel are used, but not when many other stones are used. Several cases of failure were traced to the use of dirty and unsuitable sand. The destruction of certain concrete walls of sewers and reservoirs was traced to the chemical action of sulphates, but in at least one case the concrete was both too poor and too irregular in composition to give satisfactory results.

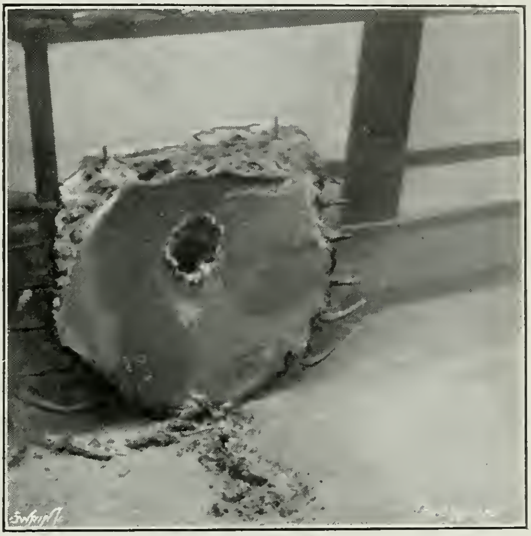
Fire tests were made on heavily-loaded floors with ballast and broken limestone as aggregates respectively; the results have been published by the German Reinforced Concrete Committee. The miscellaneous tests include many with raw materials, and a comparison of sand-lime bricks with other building materials.

A case occurred in which the tiles lining a public bath soon became loose and detached themselves. This was at first attributed to expansion of the cement, but proved to be caused by the rapid withdrawal of water from the mortar by the very porous tiles, which were evidently not kept sufficiently moist during setting.

Several interesting scientific researches are briefly alluded to in the report, and their results are to be published in due course.

Test of Reinforced Concrete with the Oxy-Acetylene Blow-pipe.—In a recent number of *The Engineer* the following account was given of a test to ascertain the resistance of reinforced concrete strong rooms :

"The question of the resistance of safes and strong-rooms of ordinary construction to the depredations of burglars armed with an oxy-acetylene blow-pipe and apparatus has, since the introduction of the latter, greatly exercised the attention of bank officials. It is a well-known fact that this new method of attacking steel is irresistible, even by the hardest metals. With a view to ascertaining the resistance of reinforced concrete strong-rooms—a method of construction which is now being substituted for the ordinary strong-rooms—a test was carried out recently with an oxy-acetylene blow-pipe and apparatus on a test slab designed and prepared by the Indented Bar and Concrete Engineering Company, of Queen Anne's Chambers, Westminster. The sample selected for the test was a slab reinforced with an indented bar $\frac{1}{3}$ sq. in. side, which had previously been used in a fire-resisting test. The concrete was composed of the following: 1 part cement, $1\frac{3}{4}$ parts sand, $4\frac{1}{4}$ parts coarse aggregate (granite).



Test of Concrete Safe Wall.

The oxy-acetylene blow-pipe was applied to the slab—the metal cutter being useless in the case of concrete—for twenty-four minutes, at the end of which period and after much raking out of the resulting glass formed by the fusion of the sand, and accompanied by the deafening roar of the blow-pipe, a hole $3\frac{1}{2}$ in. in diameter was made through the slab. Whenever a steel bar was met the metal cutter—i.e., a stream of pure oxygen directed on to the white-hot steel—was brought into action, and the steel instantly fused away. The concrete was the material which gave the trouble, the metal cutter being powerless to act upon it.

It is interesting to note that the same thickness of steel of any grade, harveyised, nickel, nickel-chrome, or any other hardened steel, would probably have been cut through neatly and cleanly by the metal cutter in about four minutes. Apparently there is no steel made which cannot be quickly cut through in this fashion, as the process is a chemical combination of the steel with the oxygen, resulting in the complete combustion of the steel. The only form of plate which offers any resistance is a combination of steel with copper or cast iron in separate sections; but even this, we are informed, does not present much difficulty to the oxy-acetylene flame.

The result of the test was as follows: Thickness of slab, 6 in.; cubic foot of oxygen consumed, ·55; cubic foot of acetylene, ·45; time occupied, twenty-four minutes; size of resulting hole, $3\frac{1}{2}$ in. diameter. The test is clearly strongly in favour of the concrete as contrasted with steel, as the time for making a hole serviceable to a burglar is practically prohibitive, to say nothing of the roar of the burner. The really enterprising burglar must clearly seek for other means of destroying concrete.

A Semi-Concrete Condenser House.—Mr. E. F. Lloyd described to the annual meeting of the Michigan Gas Association how he built what he calls a semi-concrete condenser house. The subsoil being of rather doubtful quality, and the question of economy being a desideratum, it was decided to put up a building that would accommodate itself to some possible subsidence of the ground, that would be cheap to construct, sufficiently fireproof to require no insurance, and while not decorative, yet of a generally pleasing character. A foundation wall was first built of concrete, about 12 in. thick. On the top of this wall, and before it had set, was embedded a 2-in.

BLAW COLLAPSIBLE STEEL CENTERING

CHEAPER & BETTER THAN TIMBER — MORE EASILY HANDLED — BETTER FINISH ON THE CONCRETE.

FOR SEWERS, CULVERTS, CONDUITS, ARCHES, ROOFS E^TC.

ILLUSTRATIONS SHOW CENTERS IN USE ON COMBINED BRICK AND CONCRETE CULVERT, FOR THE NORTH-EASTERN RAILWAY CO., AT BLYTH, NORTHUMBERLAND.

THE CONTRACTOR FOR THE WORK WAS MR. T. TELFORD GIBB.

THE BRITISH STEEL PILING COMPANY,

DOCK HOUSE, BILLITER S^T LONDON. E.C.

by 4-in. scantling, laid flat and set back 1 in. from the proposed face of the wall above. On this light sill were erected 2-in. by 6-in. studs of the required height, some of them being in a single piece, others, in the higher part of the building, in two lengths, spliced. These were connected together at the top with a 2-in. by 4-in. plate, similar to the sill below; 2-in. by 6-in. rough sills and heads were inserted for windows, etc.

To suitably stiffen this framework holes were drilled in the studs, and "seconds" fence wire procured from a local fence factory and run through these holes in the studs in a generally diagonal manner. These wires were then twisted tight, forming substantially a basket construction, having extreme rigidity and lightness. Thereafter forms were made, in the shape of ordinary matched pine flooring, put up in the manner of doors, long enough to span about three spaces and about 30 in. high. These were conveniently fastened both sides of the studs, and ordinary three-quarter stone concrete was filled between them, being spaded down against the forms, to bring the fine material to the surface of the walls. As the concrete set sufficiently to hold itself in place the forms were removed and placed elsewhere.

The forming of pilasters on the exterior face was accomplished in a simple manner. The various forms were held in place either by nailing to the studs or by passing wires through the forms and tightening them to hold the forms to the studs. These wires were afterwards snipped off at the wall surface. The work thus continued until the plate at the top of the studs was embedded in concrete. As the work progressed the requisite channel joists for a mezzanine floor and roof were placed.

The roof and the mezzanine floor were constructed by laying Trussit metal transversely on the steel channel joists, after which concrete mortar, made of sand and cement only, was spread over the top of the metal, the latter being sufficiently stiff to carry the workman. After the mortar applied to the upper side of the metal had partially set a plaster coating of mortar was applied below.

After the walls were completed and thoroughly dried, and the windows, etc., placed, the outside faces of the 2-in. by 6-in. studs, which, of course, are flush with the face of the concrete, were covered with vertical 3-in. by 1-in. cypress strips, chamfered, and finished in natural oil.—*The Gas World.*

Reinforced Pier Offices and Waiting-rooms at Douglas.—New pier offices and waiting-rooms are to be erected for the Harbour Board Commissioners at Douglas in reinforced concrete on the British Reinforced Concrete Engineering Co.'s system.

Mill at King's Cross, Halifax.—A new mill is to be erected at King's Cross, Halifax, for which Messrs. Jackson and Fox, of Halifax, are the architects, and in which it is proposed that the whole of the floors and roof shall be carried out in reinforced concrete.

Foundations for Retort House. — An extensive new retort house is to be erected at Nelson for the Corporation Gas Works, the foundations for which are to be carried on a reinforced-concrete raft.

Proposed Reinforced Concrete Bridge at Neepsend, Sheffield.—It is proposed to erect a reinforced-concrete bridge over the Great Central Railway at Neepsend by the Sheffield Corporation Electric Lighting Department.

Territorial Barracks, York.—In the new Lumley Territorial Barracks recently opened at York all the reinforced concrete work was carried out by Mr. W. Birch, contractor of York, under Mr. W. Brierley, F.R.I.B.A., of York.

New Arcade, Halifax. — Messrs. Clement Williams and Sons, architects, of Halifax, who have been engaged on drawings for an elaborate new arcade with extensive shops on either side running from the main thoroughfare of Commercial Street, Halifax, have now let the concrete for the whole of the reinforced concrete work to the British Reinforced Concrete Engineering Co., of Manchester.

TRADE NOTICES, CATALOGUES, ETC.

New Offices of the First Cottbus Cement Goods.—Mr. R. H. Baumgarten, representative of the First Cottbus Cement Goods and Machine Works, has informed us that the office of this firm has been removed to 8 Manor Park, Lewisham, London, S.E. Anybody interested in the manufacture of concrete bricks, tiles, pipes, slabs, posts, etc., will be shown at the above-mentioned offices how this is done on the most up-to-date methods.

Indented Bar Bulletin. —The eighteenth bulletin of the Indented Bar and Concrete Engineering Company contains, amongst other interesting matters, an account, accompanied by illustrations, of the "Afzalgunj" Bridge, which forms the main approach to the City of Hyderabad, and is the largest reinforced concrete bridge in India. Indented bars were used throughout as reinforcement.

The bulletin is, as usual, well printed and illustrated, and should serve as a useful guide to the many works carried out by this company.

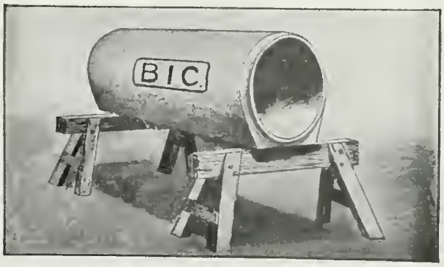
Kennedy Bar-bending Machine.—Mr. W. Kennedy, of 11 Furzeham Road, West Drayton, Middlesex, has sent us his latest catalogue of "Bar-bending Machines," which gives full particulars of his different machines and shows the method of using them. These contrivances are of the greatest possible use for reinforced concrete work, and Mr. Kennedy is also open to contract for bending bars on the site, which would in most cases be a distinct advantage.

Full information will be sent to anyone applying to the owner at the above-mentioned address.

"BIC" BRITISH "BIC"
IMPROVED CONSTRUCTION CO., LD.
TELEPHONE: 4067 VICTORIA. TELEGRAMS: "BICONCRETE, LONDON."
47 VICTORIA STREET, WESTMINSTER.

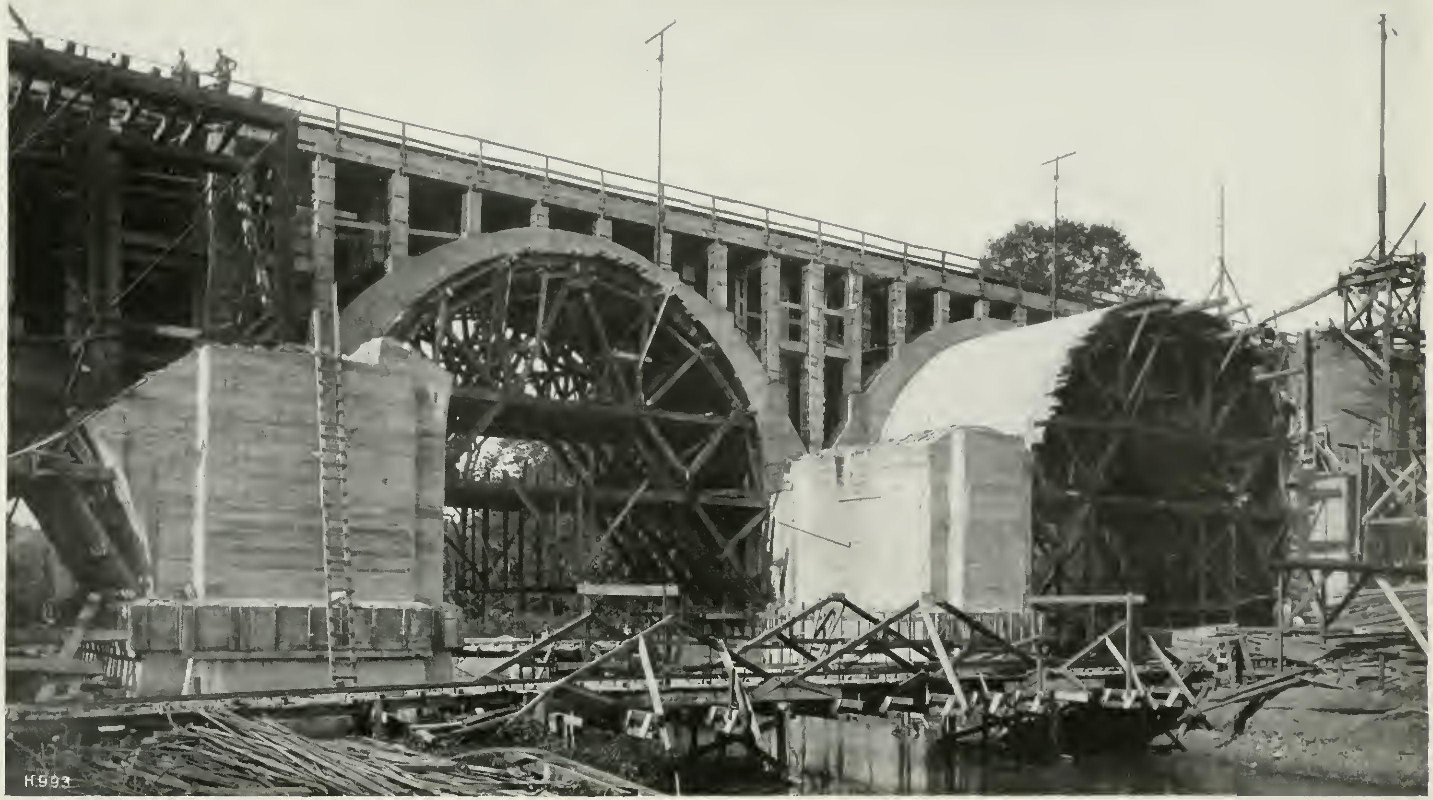
Flat-based Concrete Pipes
Three feet long to Nine feet long.

Breeze Partitions
Reinforced Sleepers
Telegraph Poles



Porous Pipes
Concrete Beams
Hollow Floors

SPECIALITY:—INVENTORS' DESIGNS CARRIED OUT TO ORDER.



View showing work in progress (*see article, page 140*)

Reinforced Concrete Bridge at Baltimore, U.S.A.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 2. LONDON, FEBRUARY, 1912.

EDITORIAL NOTES.

THE LONDON PORT AUTHORITY AND REINFORCED CONCRETE.

WE regret to hear that the London Port Authority appear to have taken a somewhat narrow view of the manner of dealing with the reinforced concrete works to be executed under their improvement scheme. They have apparently limited the design of a considerable section of the new works to one system, and have unwisely not invited designs on any other system but the one favoured, whereby again the number of contractors who have been able to tender for the work has been limited to those who happen to be licensees of the system in question, or are prepared to become licensees of the proprietors of the rights.

That the London Port Authority is using reinforced concrete is, of course, a matter of congratulation, for the use of this material spells economy, and if it is properly applied it will favourably affect the insurance rates in any reinforced concrete buildings.

Further, the system selected has stood the test of time, and the contractors who apply it or are licensees are generally of good standing.

But however excellent the policy of using reinforced concrete, and however satisfactory the system of design adopted, the London Port Authority must not make the blunder of showing partiality to any one system of design or any one group of contractors. There are many good systems of design available and many excellent contractors, and most of these proprietary systems and firms are conducted by British subjects and ratepayers of the London County.

To discriminate as the Port Authority has done is wrong, technically and economically, and apart from this there should surely be some local patriotism and some attention to the claims of firms which directly and indirectly contribute to the funds of the Port of London.

We are inclined to take the same view as we took some years ago, when a Government Department made a similar blunder—namely, that it is a blunder pure and simple. But it must not be repeated, and must be remedied as far as is still possible.

Having regard to the blunder, we trust that the modern requirements of design, factors of safety and ample fire protection will be most carefully observed, for, having made such a mistake, the London Port Authority must be prepared for encountering very watchful eyes, and ignorance of the error or the legitimate grievance in the matter cannot be pleaded after the publication of this editorial, a copy of which will reach every member of the Authority and its principal advisers.

THE REQUIREMENTS OF REINFORCED CONCRETE AND THE CONCRETE INSTITUTE.

The Concrete Institute devotes considerable time to administrative changes —its council and a sub-committee are, we understand, again engaged in this direction. In the meantime its primary objects are not being energetically pursued, and the complaints we are receiving are so numerous and frequently so bitter in tone, that we would again urge the Council to shelve its regrettable internal differences, and do some solid work in the interests of concrete and reinforced concrete. The prestige of the Institute can only be safeguarded by again accomplishing something useful promptly and well, as in its early days.

One of the most serious difficulties in the development of reinforced concrete has been the inane method of estimates and tenders which have been bad for building owners, specialists and contractors alike, and the source of infinite trouble to architects and engineers.

Cannot the Institute, with its wisdom, settle in precise terms the best practice of tenders, define the relative responsibilities of contractors and specialists, and give the professions concerned a practical lead in the matter?

We trust that the Council will turn its attention to this and other matters relating to the industry which need adjustment. We have no wish to belittle the work of the Concrete Institute of the last year, yet it would be difficult to find anything very practical accomplished.

We now publish a communication we have received on what we consider an urgent matter that should have the Institute's attention, and, in putting this letter forward, we would mention that we have several more expressing views on urgent matters that should be attended to by that body.

The regulation of the question of reinforced concrete estimates, tenders and responsibilities would be something useful. The Institute is quite as capable of doing this as the Royal Institute of British Architects was in the work of its joint committee. We trust the year will not again close without a step in the right direction being taken.

TENDERING IN REINFORCED CONCRETE (Communicated).

It is well established that, under the present circumstances, reinforced concrete specialists are put to considerable expense in the preparation of elaborate schemes and quantities in connection with all manner of tenders, so that a large proportion of the profit secured on the contracts actually obtained goes to cover the loss due to the cost of schemes either never executed or unsuccessfully tendered for.

Something should be done to alter this state of affairs, and I repeat the suggestion which has frequently been made— namely, that the engineer or architect should first prepare the general design, showing position of beams and pillars, and approximate dimensions of the various parts of the structure, then measure up the approximate amount of concrete, steel, and total centering required, or employ a quantity surveyor to do this, and afterwards write a proper skeleton specification, giving the general conditions to be observed. The amount of steel could be roughly estimated on the basis of a percentage of the concrete, as there are ample data available for this.

Having prepared these preliminary plans, approximate quantities and skeleton specification, the architect or engineer would then be in a position to get a rough idea of the cost of the work—at least to the extent of knowing whether it is worth while to go on with his scheme or whether it requires modification before the plans are sent out for tender.

Having made up his mind on this all-important matter, he can determine which of the various systems he would be willing to adopt for the execution of the work. He can thereupon schedule these in his letter inviting tenders, which letters should go to a limited number of contractors experienced in reinforced concrete work.

On receipt of these particulars, the contractor would then communicate with firms representing the various systems of reinforced concrete mentioned in the specification, in order to ascertain from them on what terms they would undertake the preparation of complete working drawings. Where special patented bars or meshwork are used, the cost of the drawings should be deducted from the cost of the steel, as the practice of to-day assumes that the cost of drawings is included in the price of the special reinforcement.

The contractors' tender, by the by, should include a schedule of prices for extras or deductions.

By the method advocated above architects and engineers would obtain tenders quicker than if they had to wait for several weeks until the specialists had time to prepare their scheme and quantities, which must of necessity be accurate, owing to the fact that, by the present method of competition, the specialist designer becomes responsible to the contractor, not only for his scheme, but also financially for the sufficiency of his quantities.

On account of the development of reinforced concrete, the specialist designers are all at present receiving a large number of inquiries which have to be studied with abnormal rapidity, with the result that they have to run considerable risks owing to possible mistakes.

It is well established that the actual method of competition tends to reduce the efficiency of the design, because the quantities have to be cut down to their last limit. This is, of course, bound to lead ultimately to the dissatisfaction both of the architect or engineer and the owner, and this state of affairs is not to the general advantage of reinforced concrete, whereas by the method recommended, once the architect or engineer has decided upon the contractor and upon the system giving the best advantages both in security and economy, there would be a perfect collaboration between the architect and the specialist designer, with the result that the structure would be better.

It may be argued that an unscrupulous designer might increase the quantity of steel and concrete to enhance the value of the work. The answer is that there are now simple means of verification which could easily be applied by the architect or engineer to check the quantities of steel and concrete.

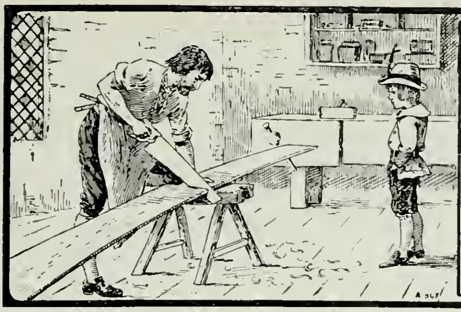
To obviate this difficulty it is suggested that in the case of specialist designers not interested in the sale of any patent reinforcement a lump sum fee instead of a commission on the value of the work executed should be fixed, and concerning the firms selling patent reinforcement it would be to their own interest to keep the cost of the tonnage required as low as possible, because this cost includes an estimated amount for the preparation of plans.

Incidentally this method of procedure would be to the advantage of quantity surveyors, who would then get the share of the work to which they are entitled. Under present conditions the specialist is obliged to prepare his own quantities owing to lack of time, and because he has to assume the financial responsibility of his quantities, he prefers to establish them himself.

As an alternative to this suggestion, of course if the architect or engineer is given a free hand by the owner he may specify exclusively any specialist or system offering the best advantages of economy and efficiency.

Regarding public works, however, it is impossible for the architect or engineer to specify any particular system or specialist designer without immediately arousing a protest from all the other specialists and also from the contractors who are not licensees of that particular system.

I am of opinion, therefore, that the only way to solve the difficulty in a satisfactory manner is that the architects and engineers, both for private and Government work, should adopt the method of procedure suggested above.



FORMS FOR REINFORCED CONCRETE FLOORS, BEAMS AND COLUMNS.

By H. KEMPTON DYSON.

In our preceding number we published particulars of a new form of centering for bridge construction emanating from America, and we are now very pleased to give an article by an English author suggesting a new method of designing forms for reinforced concrete floors, beams and columns, which merits the closest study of all interested in reinforced concrete work.—ED.

THE design of centering—or "forms" as better called—for reinforced concrete is a very important branch in the execution of the work, for not only is its erection a matter of the greatest importance as regards the cost of the work, but it materially affects the stability and appearance of the structure. Various attempts have been made to devise methods of centering which shall possess features of economy and efficiency. It seems to be accepted by many contractors, however, that it is hopeless to attempt to devise a standard centering, specially in the conditions of English practice, where many buildings are situated upon awkwardly-shaped and restricted sites, which result in beams and panels being of very variable dimensions.

Where a building is regular in disposition and, both by reason of the purpose it is to serve and regularity of site, the floors are designed with equal-sized floor panels and beams, it is possible often to design centering applicable for that job which can be used over and over again, and in such cases the timber is made to serve many times over, some attempt being made to design a centering which shall be easily removable, and that can be easily re-erected. In such cases the work becomes much more economical and efficient.

But, as has been stated, the conditions of the majority of English buildings are against maximum economy being obtained in this way. Therefore, when floors have all differently-sized beams and floor panels, reinforced concrete is placed at a material disadvantage because of the greatly increased cost of the centering which is so important to the work. The cost of the centering may vary from, say, 10 to 70 per cent. of the total cost of the reinforced concrete work, and, even under fairly good conditions, frequently comes out at 30 per cent. of the work. Reinforced concrete design is economical with regard to the concrete and steel required, though the labour is generally more than with ordinary structural steel and concrete floors. However, if it were not for the centering in almost all cases reinforced concrete would be found more economical than ordinary standard steel sections and concrete filled between, and could be carried out with as much speed.

If, therefore, it were possible to devise a method of centering which could be employed in the majority of cases in England and for almost any job, and which offered, if not an ideal standardised system, some features of economy, it would lead to the greatly increased use of reinforced concrete, and would in some ways help to bring the material into better repute, because, when the reinforced concrete contractors are so anxious to prove the superior economy of their material over ordinary steelwork construction, there is a tendency to cut the centering too much and risk bad workmanship. The problem is, consequently, one which may well exercise the attention of contractors and others. Most contractors work with one or two specialist designers, and a standard centering could be devised and worked to by both the designer and the contractor.

The suggestion here put forward is not claimed to be above reproach; it is only brought out as a record of some ideas which may possibly be developed and improved in practice, or, perhaps, if not considered suitable for adoption in any respect, may lead to something really serviceable being devised. The conditions which it is attempted in this proposal to fill are (1) to have a number of parts in centering which can be kept in stock by general builders or by specialist contractors for reinforced concrete work, these parts being such as will work in frequently for jobs of very varied character, so that the cost will be spread over a great amount of work; and (2) that where timbers have to be cut to length they shall be kept in as long lengths as possible, and not cut about so that they cannot be used again except for such purposes as firewood.

One finds in the majority of cases that after each job contractors have a great deal of timber in short lengths, which is never worked in again, and is ultimately made use of for firewood, a small amount, perhaps, being used for such purposes as filling boards, and so on, where short lengths are usable. If only the timber could be kept in fairly long lengths, though it had to be cut about somewhat, it would be usable many more times than when the framing of the beams and floor panels is, as is often seen, cut into short lengths to merely fill in between, giving an appearance of patchwork. Not only does one see this in beams and floor slabs, but often in column forms and in uprights or props.

In attempts at producing a sort of standard design of centering a feature has often been the tendency to economise props. Now, considering that the props are the most usable pieces of timber, and undergo only a comparatively small amount of wastage, this is no particular advantage, although it is urged for the policy that it leaves an unencumbered space in the floor area below. One can, however, obtain a sufficiently unencumbered space below even when a large number of props are used. These props are, of course, of heavy timber, but that timber, if it can be used over and over again, does not add much to the cost of any one job.

These preliminary remarks will enable us to describe the design here put forward, and call attention to the manner in which the above-named *desiderata* are to be met.

Fig. 1 shows the manner of framing the forms for supporting floor panels and secondary beams, *Fig.* 1 being a cross-section through the secondary beams, and *Fig.* 2 a variation of the framing of the secondary beams.

Firstly, it will be noted that the centering is arranged for a secondary beam measuring as a maximum 6 in. in width by 10 in. in depth, this being reducible in depth by the aid of blocking pieces, as shown in *Fig.* 2. If a designer chooses to make the width of his beams 5 in., say, which is about the minimum which they could be made, or prefers not to go below 7 in. or 8 in.,

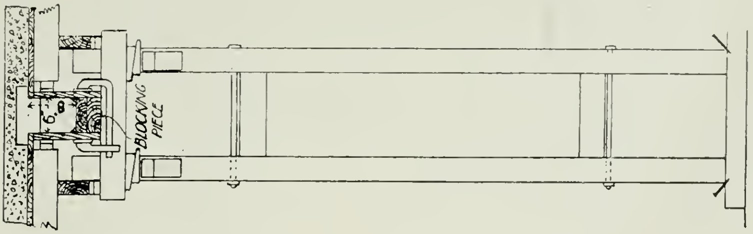


Fig. 2. Alternative.

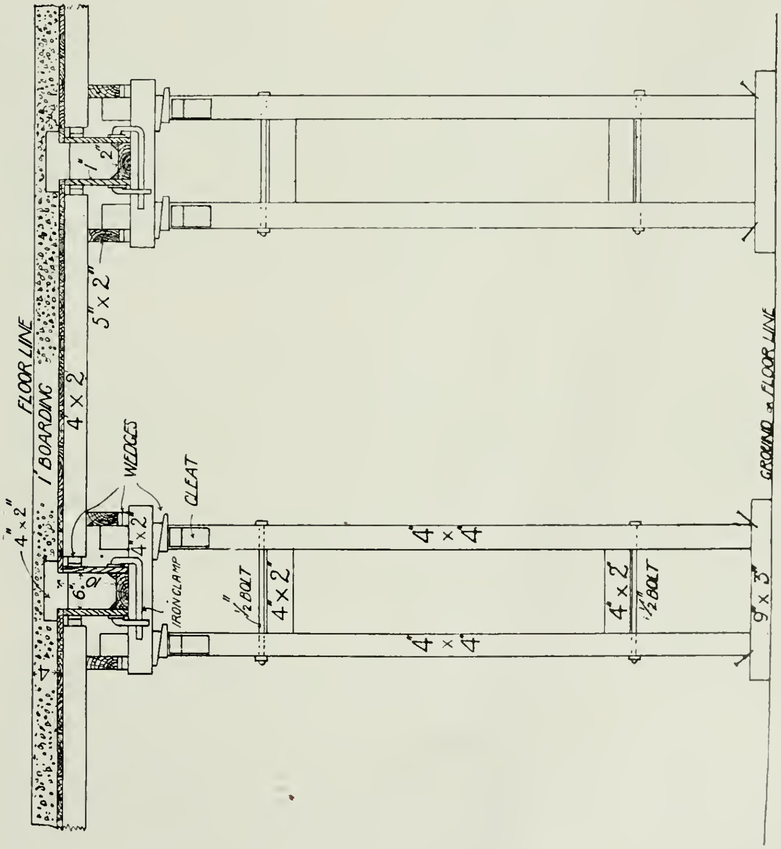


Fig. 1. Secondary Beam Form.

Scale ⅜ *in.* = 1 *ft.*

the modification is easily made; but 6 in. for the width of a secondary beam is probably the best as a standard width, and the depth is the dimension in which the variation will probably arise.

The sides of these beam boxes are framed of 1-in. stuff for cheapness and convenience in handling. The bottom of the beam box, however, is made of 2-in. stuff, because it has to carry weight over some distance without sagging. The beam boxes are sustained on double uprights bolted together, which extend up on either side of the beam boxes, adjusted in height by folding wedges at the tops, driven in between clamps (which latter are screwed to the uprights) and the cross-bearers under the bottoms of the beams. These cleats can easily be attached at any height if such alteration be necessary, and the only material damage by such alteration would be in the destruction of an odd piece of timber such as the cleat would be made of. A different size of base-plate at the bottoms of the uprights may also be employed to vary the height. The sides of the beam boxes are clamped together at the bottom by iron clamps, a suggestion for which is given in *Fig.* 3. By reason of the inclined hole in the shorter loose end, these clamps wedge themselves in place, and are easily removable by just tapping the short end. The cross-bearers in the bottoms of the beams sustain 5 in. by 2 in. joists, which again carry 4 in. by 2 in. joists, these in turn sustaining 1-in. boarding, which receives the slab concrete. These do not extend completely between the beam boxes by more than 2 in., and therefore a variation in length is not material, so that the bulk of such timber kept in stock would work in again without cutting.

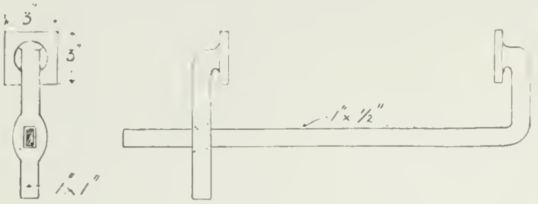
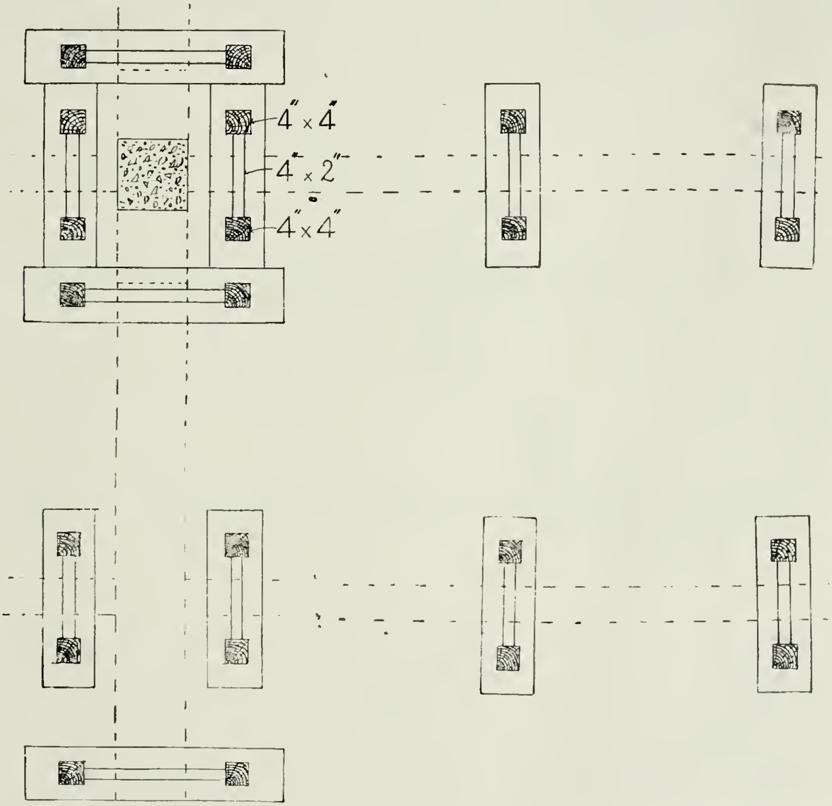


Fig. 3. Clamp.

The tops of the beam boxes are kept in position by wedges in between these 4 in. by 2 in. joists, and the 5 in. by 2 in. joists are wedged up to the correct height off the cross-bearers, which in turn are wedged from the cleats on the uprights. The beam boxes are kept open by a 4 in. by 2 in. blocking piece as shown, these being taken out when the concrete is filled in the beam and, by its own weight, serves the purpose of keeping the boards tight against the external wedges. Now, it is easy to see that such a form of upright is preferable, as they are not underneath the beams, which would require them to be of one height, and would necessitate frequent cutting in buildings with floors of different heights.

The boarding to sustain the slab concrete, it should be noticed, is made to run parallel with the beam boxes; this means that the boarding is not cut into short lengths to frame between the beams, but can be used in fairly long lengths, an economy which has been referred to above. The end boards, which may overhang the 4 in. by 2 in. joists, bear upon the tops of the side boards of the beam boxing, and are the only fillers, and a few narrower boards may even then be used to avoid cutting. The boarding, being 1-in. stuff, is cheap and readily handled, and is therefore very economical, but the supports have to be placed fairly close together to prevent any sagging, a point which wants careful watching, because where such light boarding is used one often finds the work very uneven, and this unevenness often necessitates rendering, which adds so greatly to the cost of the work.

This design fulfils another requirement, which it is worth while endeavouring to meet, and that is that every part should be able to be removed independent of the other, and allow the wedges to be removed, firstly, so as to observe if the work is defective, as in some cases it is only possible to see that the work is satisfactory by removing the uprights of the centering, which risks the sudden collapse of the whole structure. Thus, in this particular portion of the work, it is possible to firstly remove the sides of the beams, which should be done in two or three days, so as to allow the air to dry the concrete and harden it. It is then possible to remove the wedges from under the 5 in. by 2 in. joists and turn over on either side the 4 in. by 2 in. joists, so stopping the boards from tumbling down and injuring the men removing the centering. These boards can then be taken away, still leaving the 4 in. by 2 in. joists in place. A few days later, if this be the method adopted, the wedges in between the cleats and cross-bearers of the beams can be eased, when the whole of the weight will begin to bear on the beams, and one can observe the deflection, and see whether the work is good; should there be any failure evidenced, the slabs would be caught by the 4 in. by 2 in. joists, which have been turned over on the flat, and by the bottom boards of the beam boxes.



Scale ⅜ in. = 1 ft.

FIG. 4. PLAN OF UPRIGHTS.

Such a design, therefore, as suggested, fulfils all requirements; the parts can be removed separately with safety, the work is not damaged in the removal, the timbers can be used again without much cutting, and the labour of removal and re-erection is small.

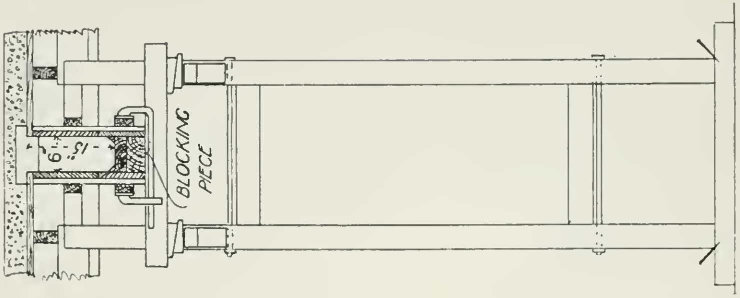


Fig. 6. Alternative.

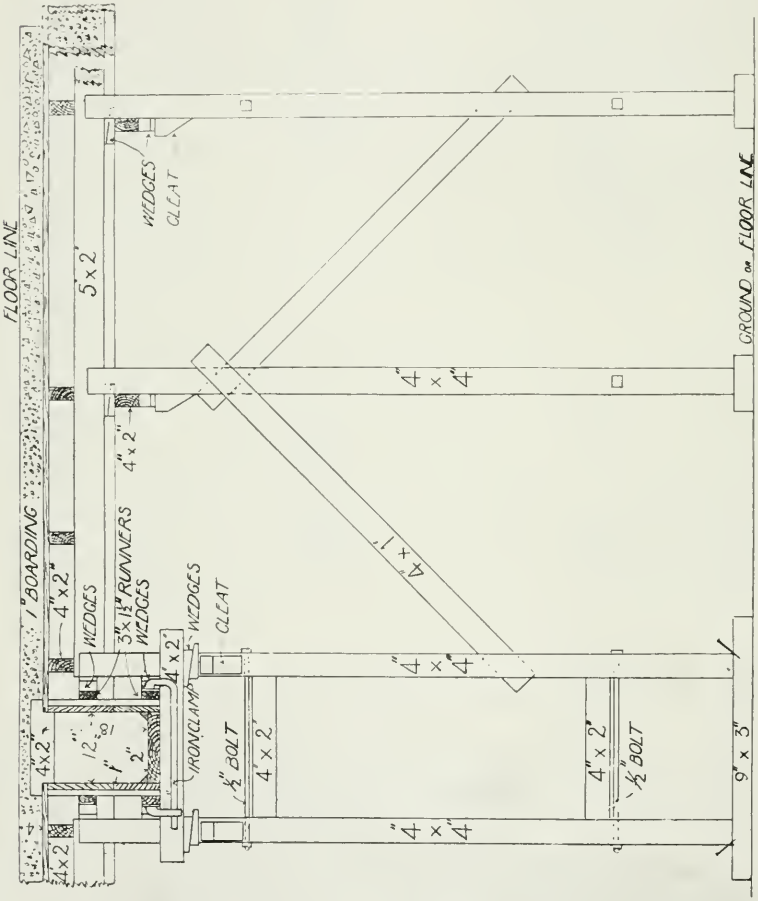


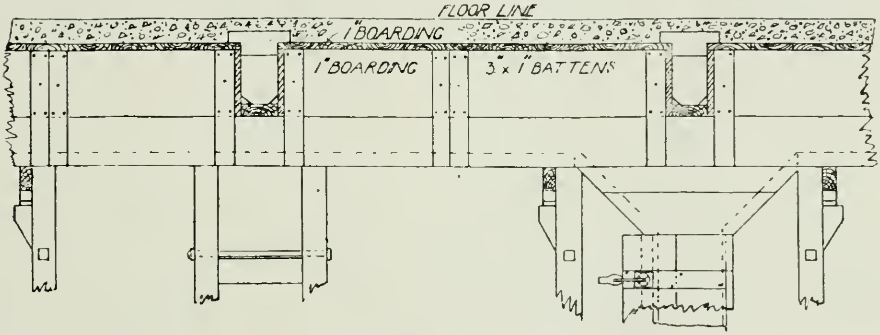
Fig. 5. Main Beam Form.

Scale ⅜ in. = 1 ft.

The 4 in. by 2 in. blocking pieces at the tops of the beams are also serviceable to suspend the steel the right distance above the bottom of the beams. The depth of the beam is accurately gauged by nailing the triangular fillets the right height above the bottom, and then wedging up the 2-in. bottom boards to stop against them.

The standards are placed convenient distances along the secondary beams, as shown in the plan, *Fig.* 4 and in *Fig.* 5, and are cross-braced diagonally to give stiffness, these cross-braces being 4-in. by 1-in. boards, tacked by nails. These nails, as all others, it should be noted, should be left with their heads projecting, so that they can be easily withdrawn.

The sides of the secondary beams are formed of long lengths of boards, and, working to the standard size suggested—namely, 12 in.—one can form the sides of the beam boxes of two 6-in. boards. These boards need to be cut to length, but they are of sufficient length, as a rule, to enable them to be worked in again, or, even should they be eventually cut into short lengths, it is possible to use them for filling boards, as will be detailed later. The ends of the secondary beam boxes, of course, have eventually to meet or enter the main beam boxes. The advisability of using the 12-in. boards as the sides of the beam is again apparent, because the size is best to work in with the framing of the main beam boxes. The size, however, of the work can be varied by the use of different size boards, and yet the same method of framing would be advisable—that is to say, supposing one wished to have 14 in. as the minimum depth of the secondary beam, two 7-in. boards might be used to give this depth. Thus, the main beams would be framed up with three 7-in. boards. *Fig.* 6 is a detail of the framing of the main beam boxes, and shows how the secondary beam boxes enter. This diagram will, however, be referred to more in detail in describing the main beam boxing.



Scale ⅜ *in.* = 1 *ft.*

FIG. 7. MAIN BEAM FORM.

The same principle of adopting a maximum size for the secondary beams is adopted in connection with the main beams. In *Fig.* 5, the main beam is shown as being 12 in. by 18 in., internal dimensions, and *Fig.* 6 shows how, by putting in a different width of bottom board, and putting a blocking piece over the cross-bearers which rest on the uprights, this is reducible to a beam of 6 in. width, and 15in. more or less in depth. The plan shows how the uprights are placed for the secondary beams and the main beams. The main beams vary not only in height, but in width, whereas secondary beams do not vary, say, in width. It, therefore, means that the bottom boards have to be made up to the width, and in the scheme here proposed, it will be seen, from *Fig.* 5 and *Fig.* 7, that both the bottom boards and the lower side boards are in long lengths, and therefore entail little cutting to waste. The special feature of the upper portion of the beam boxes is the use of filling boards, framed together in standard lengths. Details of these filling boards are given in *Figs.* 8, 9 and 10, which show three standard lengths. The sizes which it is suggested should be kept in stock are as follows :—

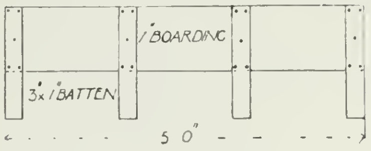


Fig. 8. Filling Board.

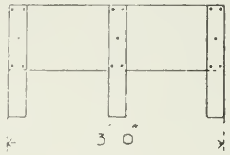


Fig. 9.

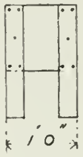


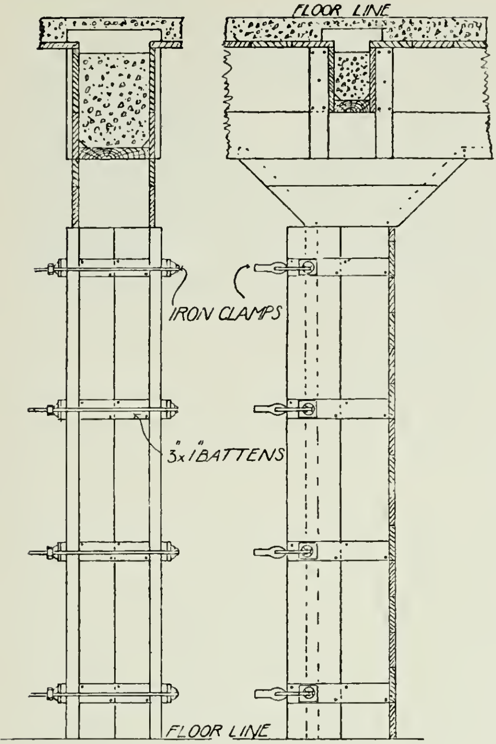
Fig. 10.

Standard Lengths.

Feet	Inches.	Feet	Inches.
1	0	2	0
1	1	2	6
1	2	3	0
1	3	3	6
1	4	4	0
1	5	4	6
1	6	5	0

The boards are framed up about 12 in. deep, as suggested above, for the reason cited, made up of two 6-in. boards, cleated together by 1-in. by 3-in. battens, as shown, with projecting ends. Such stuff would be taken from stock, and a builder could easily keep a number of these in stock, instead of making firewood of such material; much economy would come by using his short pieces for these filling pieces. *Fig.* 7 shows how, in between the secondary beam boxing, the spacing for which happens to work out that way, two different lengths of filling pieces are placed. By arranging the different standard sizes, one can make up any width with exactitude. The secondary beam framing, it will be seen, is sustained quite independent of the main beam boxing. Along the side of the beam boxes, to bear against the battens that cleat the filling pieces, it is suggested that 3-in. by 1½-in. runners should be placed to take wedges. Where the runners occur, but between the uprights, iron clamps similar to those above described are used to clamp the bottom of the boxes. Other details are very similar to the secondary beam centering, and the sides of the beams are removable earlier, to provide for the drying and hardening in the greater thickness of the beams before striking the other part of the centering, the advisability of which has already been referred to.

The upright shown for sustaining the main beam boxes is slightly higher than that for the secondary beams, but it is just to point out that the length of the standard could vary, though they might have all been of the same height. The uprights generally are framed out of 4-in. by 4-in. timbers, with distance



Scale $\frac{3}{8}$ *in.* = 1 *ft.* Fig. 11. Column Form.

pieces of 4 in. by 2 in. just let into mortises, the uprights being tightened against them by $\frac{1}{2}$-in. bolts, as shown in *Figs.* 5 and 6. The 4-in. by 4-in. stuff is thought to be preferable to the use of round scaffold poles—or, rather, short lengths of same—which are often used by contractors in rough centering. They could be adapted, but it is thought that the 4-in. by 4-in. stuff is as cheap, and has sufficient superior advantage in its squareness to warrant its use, and, of course, some contractors would find an insufficient supply of round timbers by the inevitable wastage of scaffold poles. Often, 9-in. by 3-in. deals are used for the purpose, and, indeed, any handy stuff is made to serve, but they would not do for a standard design.

The 9-in. by 3-in. base-plates to the uprights are employed for the purpose of distributing the load over the ground or floor over which they are strutted. There have been several mishaps through the settling of supports in soft ground and the concentration of a load on a small area of an insufficiently hardened floor below. This should emphasise the necessity of always providing such base plates. The uprights are secured against any possibility of slipping by means of the nails driven crosswise into the base plates.

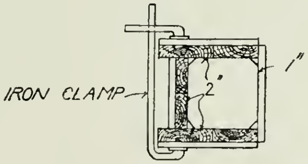


Fig. 12. Small Column Plan.

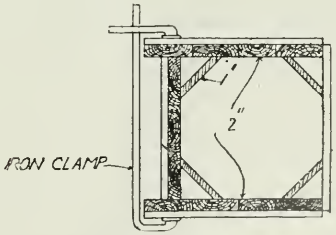


Fig. 13. Large Column Plan.

The uprights for the main beams, as shown on the plan, *Fig.* 4, would obviously have to be wider apart than the uprights for the secondary beams. Such uprights would be placed as close together as found necessary, by reason of the load to be sustained. The supports, too, would be placed all round a column, as shown, so as to permit the column boxing to stand quite independent, and allow it to be removed before striking the centering of beam or floor slabs, it being desirable to remove the column boxes (the same as the sides of beam boxes), in order to allow the greater thickness of concrete to harden by the drying out of the moisture by the circulation of air, which would otherwise be prevented if the boxing remained in place.

Figs. 11, 12 and 13 show details of a suggested standard form of column boxing, the sides of which are held together by the clamps, of which detail is given in *Fig.* 3. *Fig.* 12 shows how to frame up a small column, and *Fig.* 13 a large column. In both it will be seen that three of the sides are framed up firstly with 2-in. boards placed vertically, and the fourth side is brought up bit by bit by nailing 1-in. boards horizontally across. When several boards are required to make up the width of the sides, battens are nailed across to keep the boards together. Triangular fillets are easily tacked into the corners of the columns to enable the centering to draw away without damaging the corners. In large columns an hexagonal section is often adopted, for which the design shown in *Fig.* 12 is suitable. Such columns should, of course, have hooped reinforcement in them. The beams are splayed down on to the columns, and such splaying is usually provided for, as shown in *Fig.* 11.

On some jobs in America a box-like form has been framed up, in which the whole of each panel, and the sides of the beams, are framed up to form a sort of box, which is constructed with a hinge or joint in the centre, so as to come away easily. On striking, the whole box is dropped and lifted up by a crane, cleaned, and put into a new position. As, however, with such a type of centering the several floors need to be exactly alike, it is not likely to generally fulfil English requirements, and it is thought that the design here put forward is superior in many ways. Of course, if the job is a large one, and the planning is very regular, and the sides of the beams and floor very uniform, then it would be easily possible to devise a form of centering for that one job which would be more economical than the design here put forward; but it is not claimed that this design is the best for all conditions; it is only a suggestion of a form of centering which might be adopted for average conditions with some advantage over the ordinary unsystematic methods of centering in vogue, the wastage under which seriously retards the development of reinforced concrete in many directions.

THE REINFORCED CONCRETE SPECIALIST.

By LUCIEN SERRAILLIER, M.C.I.

The following article on the subject of the position of the specialist in reinforced concrete merits attention, and we shall be pleased to open our columns to further articles or correspondence on this subject.—ED.

The reinforced concrete specialist, a product of the last twenty years in this country, is the logical outcome of conditions which must necessarily arise from the introduction not of a new branch of engineering, but of a new material and method of construction which is applicable to the whole range of engineering and architectural work. And herein lies his *raison d'être*, for he has come not to oust the professional engineer or architect from his legitimate work, but to act as his ally. The architect's sphere is still the design of beautiful and appropriate buildings, and his reliance upon the specialist for the structural portion of the work is but the continuation of the policy he has followed in the past of reliance upon the structural steel specialist for the design and calculation of the girders, joists, stanchions, a form of construction which is now being rapidly superseded by reinforced concrete. It is this intimate association between art and science, between architect and specialist, which appears likely in the next few years completely to revolutionise our buildings, both in external appearance and internal disposition.

Reinforced concrete has possibilities, structural, artistic, and economic, which far exceed any possessed by steel construction. We have long since reached the limitations of the latter, whereas the economic and structural advantages of reinforced concrete are only now being borne in upon us, and we are but on the threshold of what may be accomplished in the outward expression by form and colour of this new material. Here is the architect's opportunity, and far from a conflict of interests between him and the specialist, we discover rather a community of interests in the design and execution of structures which combine beauty with utility and practicability.

It is, of course, quite customary to specialise in engineering work. The engineer, after a general training and experience, frequently confines his activities as an expert to dock or railway work, to sewage, waterworks, or other of the many branches in the whole range of engineering practice. This practice is also immune from the intrusion of the reinforced concrete specialist, who does not encroach upon any of these departments, but merely specialises in what may become an adjunct common to all these branches, his function being merely to apply a new material to all classes of construction. The specialist, for instance, will design a reservoir, but does not profess to scheme out the water supply of a community, which is strictly within the province of the water engineer, whilst that of the specialist in this instance is to apply his knowledge to merely part of the general scheme. And so on with every class of engineering structures. For instance, the consulting engineer of a trading concern has to deal with the water supply, the lighting and ventilation, and erection of machinery, as well as the construction of buildings. He must perforce consult these highly specialised branches of engineering, and there is no reason why he should not in like manner employ the services of a reinforced concrete specialist for the structural part of the enterprise. Thus we again possess all the elements of an alliance instead of a conflict of interests between engineer and reinforced concrete specialist, which certainly permits the former to consult the latter without derogation to his professional status. The function of the engineer may well be the co-ordination of the knowledge and experience of these various specialists rather than an attempt to know in detail every branch of engineering, an attempt which must necessarily fail.

The specialist, however, is still regarded by many as an intruder, and when he combines with the offer of his designing knowledge and experience the sale of some reinforcing material he is apt to be considered in the light of an ironmonger. But these prejudices, arising chiefly from ignorance of the true function of the specialist, are rapidly disappearing, if one may judge by the amount of work which is now flowing into the offices of the various specialist firms established in this country.

There can be no two opinions as to the indebtedness of the community to the specialist firms who have been responsible for the introduction of reinforced concrete into this country. Had it not been for them the natural conservatism of both engineering and architectural professions would have delayed matters for many years. Experiments are costly, especially in civil engineering, where everything is on a large scale, and the very natural attitude of the profession, which has everything to lose in the way of reputation and practice by the introduction of an untried form of construction, was one of caution and maintenance along lines of proved safety. The question naturally arose, who is to be responsible for work done in accordance with this new method of construction? The specialist stepped bravely into the breach and backed his opinions by deeds, by the acceptance of financial responsibility. One may well lay stress upon this point, because the specialist has in most cases been criticised for confining the permission of constructing on his designs to a limited number of contractors, and thereby restricting competition. Such action was quite natural at the outset as a protection against a financial responsibility thrust upon him which no private professional firm would accept. This security was assured by restriction to a few approved contractors experienced in carrying out his designs, and, however obnoxious such an approach to a Trust may be to the average Englishman, there is no doubt this wise policy has prevented disasters which would have retarded the industry for many years.

This limitation of contractors is now not only tending to disappear, but bids fair to drift to the other extreme, a system of unrestricted competition, with which it is now proposed to deal.

The opportuneness of the advent of the specialist and the services he is able to render to both professions will probably be generally admitted. The manner in which such services may best be utilised to mutual advantage is, however, a matter of some contention.

Briefly, two classes of reinforced concrete specialist firms exist: those which exploit patent systems, and those which sell patent bars or reinforcing material. The former derive their profit by charging a percentage on the value of a contract for their professional services, the latter by charging such a price for the bars they sell as will afford a profit equivalent to the percentage they also require for their professional services. This article does not purpose to deal with the relative merits and disadvantages of these two methods; each has its camp of adherents, but that they both render services is testified by the increasing work which is placed with them. This divergence is negligible, and is principally a matter of a slight difference in name. What is of importance is what is the best way in which to take advantage of the services offered by either class of specialist.

The general practice for the engineer or architect who has a scheme to be designed in reinforced concrete is to invite tenders from all and sundry, under the firm impression that an open competition will produce the best economic and structural results. With the object of obtaining the best work at the cheapest price, leading systems are invited, and each system has its array of contractors; in some cases the same contractor may tender on different designs from two or more competing specialist firms. It appears, however, that this object is not achieved, for in the first place it is generally difficult to adjudge the contract to any other than the cheapest tender, which is no criterion that the best design, which is of primary importance, has been obtained. Nor is the successful tender necessarily as economical as it might have been, for it certainly does not follow that the cheapest tender accepted could not have been cheaper had the same contractor tendered on the best design if the latter had been selected beforehand.

The element of chance enters too largely here, for the specialist firm which has submitted the best design may have been unlucky in the tenders of their contractors.

Again, the specialists who are invited to submit designs in competition with one another are all doing the same work, in which only one can be successful. The non-successful ones, therefore, are losing time and money. Specialists must live, and, since they thrive, it stands to reason that they must make good their loss in one direction by profits in another—that is, on jobs in which they are successful.

This remark applies equally to contractors, and, in fact, to all competitive work which is carried to an extreme. The problem is to strike a happy mean between indiscriminate and unrestricted competition and privileged monopoly. The mistake lies in mixing up the question of design with the question of tender.

The remedy is to select the specialist firm and system to be adopted and arrange their fee beforehand; or, if competition of design is desired, let it be limited to two or three specialist firms instead of allowing a job to be hawked about, which has a discouraging effect in the long run on competing firms. Having selected the design, the client should then, and only then, invite tenders from contractors on a proper bill of quantities. It is a well known fact that there is very little difference between the well-considered designs of various specialists for the same structure. The question of price, therefore, depends very largely upon the contractors, and it is conceivable that the best and most economical design may have been discarded because the tenders upon it happen to be too high. The tenders from one contractor on several competing designs for the same job do not show anything like the variations in prices that is shown by the tenders from several contractors on the same design. Hence the proper procedure is to select the design first, and then invite tenders upon it, instead of jumbling up the two essentially different processes as at present.

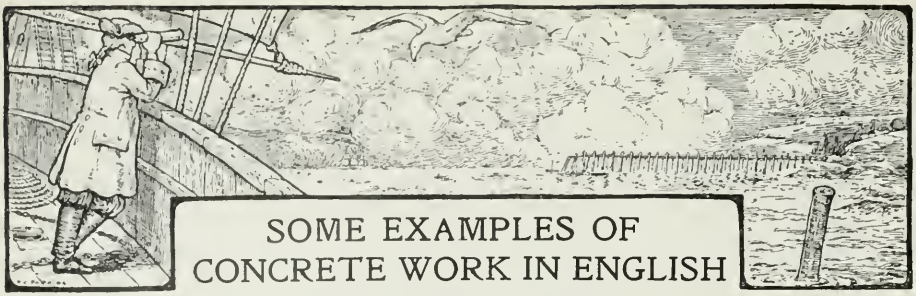
Another opportunity for improvement lies in extension of time for the preparation of plans. As a rule the time given is too short, and the result is hasty work which may contain mistakes which lead to friction between client, specialist and contractor. Too little consideration by a client is generally shown on this point, and the practice, hardly consecrated by justice or crowned by success, of attempting to hold the specialist responsible for the accuracy of the quantities of concrete and steel, does not conduce to economy; for the specialist, if pressed for time, will generally err on the generous side of his quantities rather than run the risk of having to make good out of his own pocket the deficiency due to under-estimating. Here, again, the client pays in the long run.

The method of payment to the specialist also leaves much to be desired. It should be made direct by the client, and not through the intermediary of the contractor. The specialist is selling his services, and this has nothing to do with the financial stability of the contractor, upon whose selection he generally has very little to say. For the contractor to pay the specialist is to deprive the latter of the control he should, on the contrary, hold over him. This consideration brings one to a recent departure which has occurred in the contracting world. Some contracting firms have recently joined the ranks of specialists by organising a designing department in conjunction with their ordinary business as contractors, such departments being generally run by engineers who have acquired their experience among the original specialist firms. Such an arrangement is not desirable, since it attempts to assimilate two separate elements—execution and supervision. It is difficult to conceive how it can benefit any but the contracting firms in question, whilst from the client's point of view it is absolutely disadvantageous. The position of a contractor who tenders on his own design is anomalous. The prime object of a contractor is to obtain the contract, and he knows that price tells. The necessity of control is, therefore, all the more imperative when, as in this case, there is a temptation to cut things fine. For whether the client be professional or otherwise, the fact that he invites designs from a specialist implies a certain ignorance of the subject which, in the case of such a highly specialised subject as reinforced concrete, is not derogatory to his professional capacity, as has already been pointed out, but which undoubtedly weakens his control over the actual construction, if the designs have been prepared by the contractor. In the method suggested, whereby the specialist is first selected, the necessary control over the executers of the work —that is, over the contractor—is either shared by client or specialist, or at least delegated by the former, who still retains the ultimate power, to the latter. There is no doubt that suitable provision should be made for supervision of a reinforced concrete contract by the specialist, but when the contractor is his own specialist and supervisor he is beyond control and criticism, and the position becomes not only illogical but extremely dangerous.

Considerable expense on the part of specialists would be saved if clients would only limit their invitations for designs and tenders to jobs which are practically certain to mature. The nebulous schemes which are presented to the poor specialist—schemes which have the most slender chances of materialisation—only go to swell the expenses of those firms, who must necessarily recoup themselves for this loss by increased profits in other directions.

The attitude of the Local Government Board on the subject of reinforced concrete is both incomprehensible and illogical. Every other Government Department has recognised this new form of construction. Despite the numerous public buildings which are being erected everywhere, despite the proof of solidity, economy, durability, and reliability which has been shown by reinforced concrete in structures which have stood the test of a quarter of a century, the L.G.B. generally refuses to authorise loans on the same terms as masonry or even timber structures. Reinforced concrete, it would appear, is still in the experimental stage, and the attitude of the L.G.B. remains Early Victorian. Time, of course, is the solution; the younger generation of advisers cannot fail to give reinforced concrete the same recognition it has received in all other quarters—commercial, professional, and official. The long-suffering tax-payer is consequently called upon to pay for the more expensive forms of construction in the case of nearly every job in the country which is brought to an untimely end by the too stringent financial conditions of the L.G.B.

The future of the industry is, however, full of promise. A more thorough acquaintance on the part of the public with the conditions which surround the business of the specialist cannot fail to improve these conditions, possibly on the lines which have been briefly indicated in this article; and the attitude of a Government Department, based on obsolete traditions, cannot long continue to hamper the development of a form of construction which has received universal recognition.



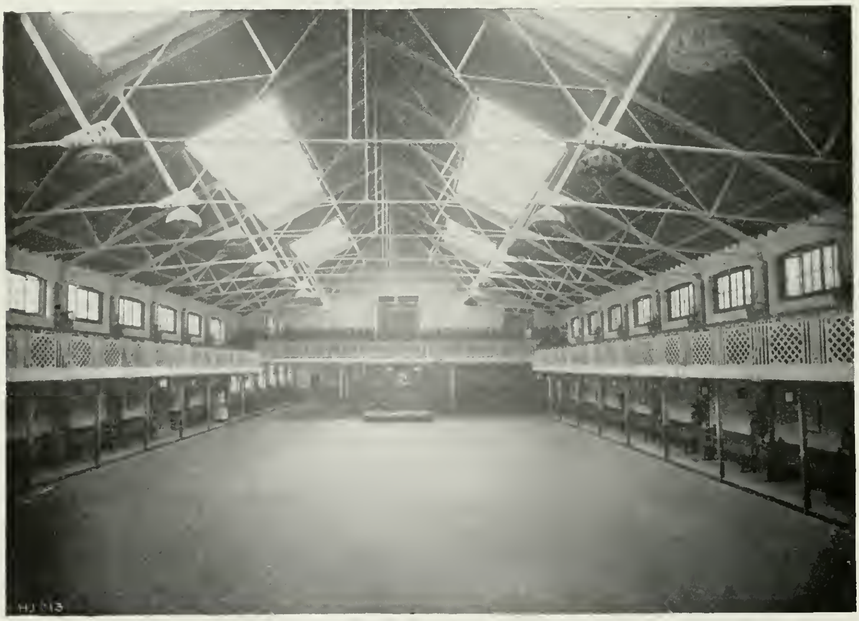
SOME EXAMPLES OF CONCRETE WORK IN ENGLISH SEASIDE TOWNS.

We are pleased to observe the increased use of reinforced concrete in the construction of large places of amusement and similar buildings in our leading seaside resorts, and the following article, giving illustrations of a few of these buildings, should be of interest to our readers.—ED.

REINFORCED CONCRETE WORK IN THE NEW KURSAAL, WORTHING.

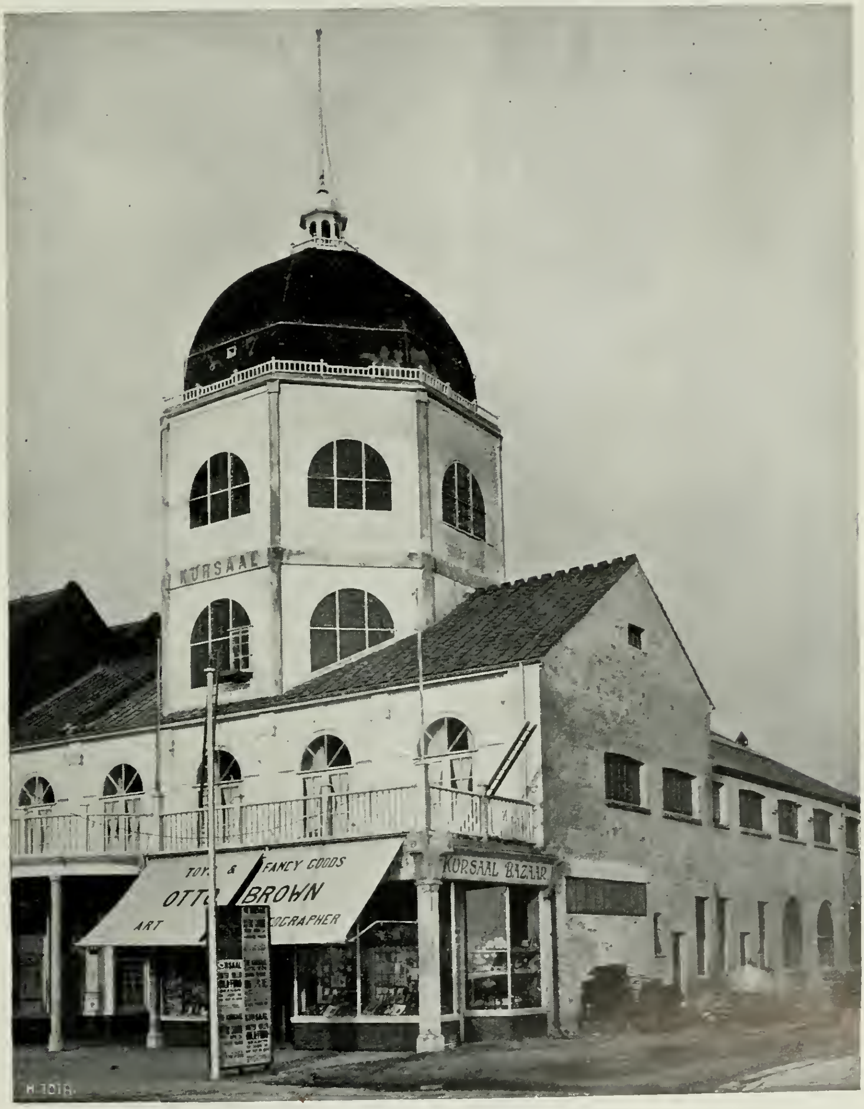
REINFORCED concrete has been largely used in the construction of the new Kursaal at Worthing, a view of the interior of which is shown on this page.

The building is a steel-frame structure. The walls of the large hall are of brickwork panelled between R.S.J. stanchions, while the walls of the tower are of concrete reinforced with Expanded Steel. Further, the suspended flooring in the tower and its dome are of concrete reinforced with Expanded Steel, and the suspended flooring in the main building and the steps and risers and suspended flooring to the galleries are also of concrete reinforced with Expanded Steel.



Interior of the Kursaal.
REINFORCED CONCRETE WORK IN THE NEW KURSAAL, WORTHING.

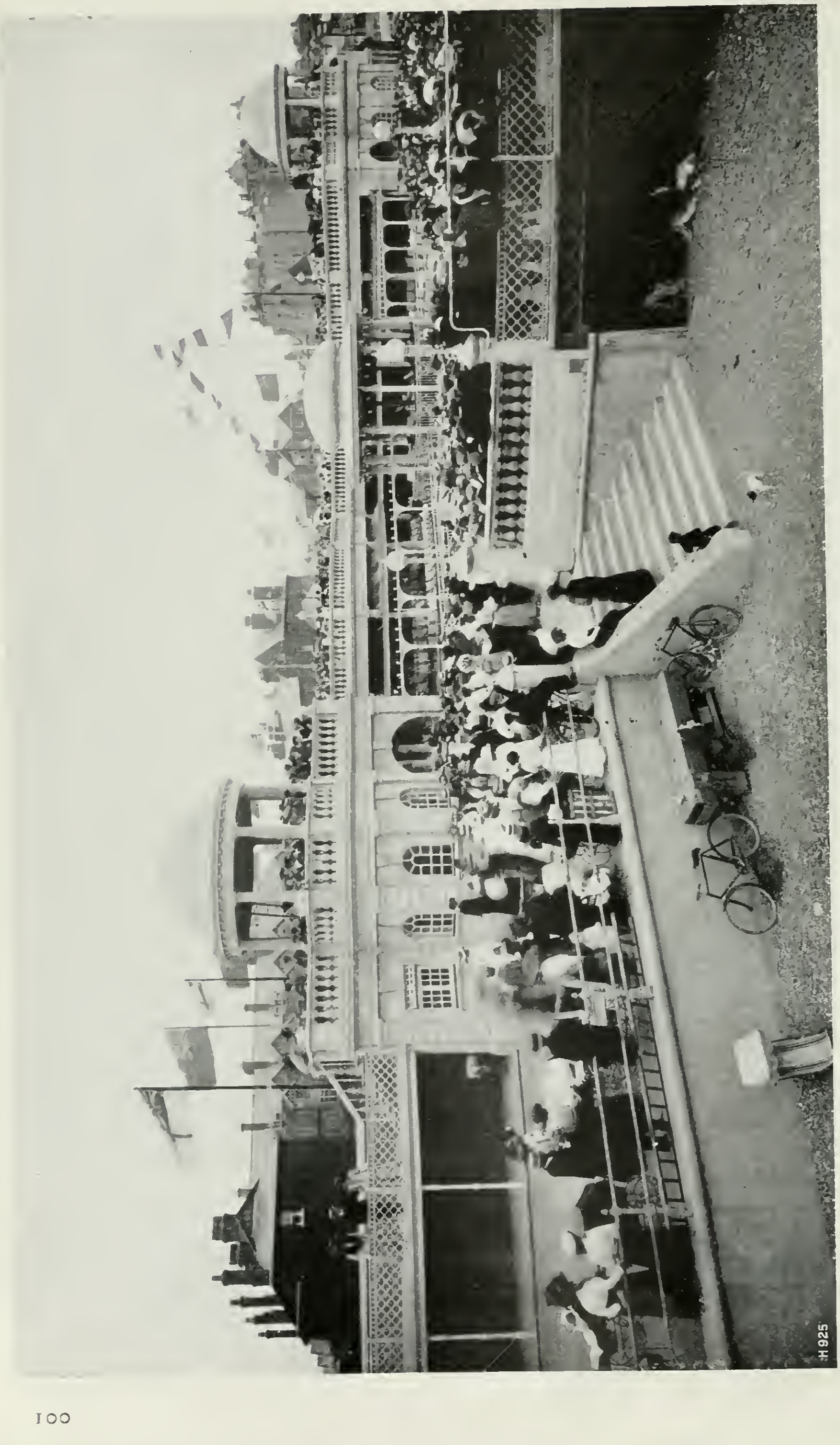
The work was carried out to the designs and under the supervision of the architect, Mr. T. A. Allen, of 3, Duke Street, Adelphi, London, W.C. The steelwork was supplied and erected by Mr. John Every, Phœnix Ironworks,



View of finished building.
The New Kursaal, Worthing.

Lewes; the reinforced concrete and other work being carried out by Mr. A. Crouch, contractor, Montagu Street, Worthing.

The reinforcement throughout was supplied by the Expanded Metal Co., Ltd., of London and West Hartlepool.

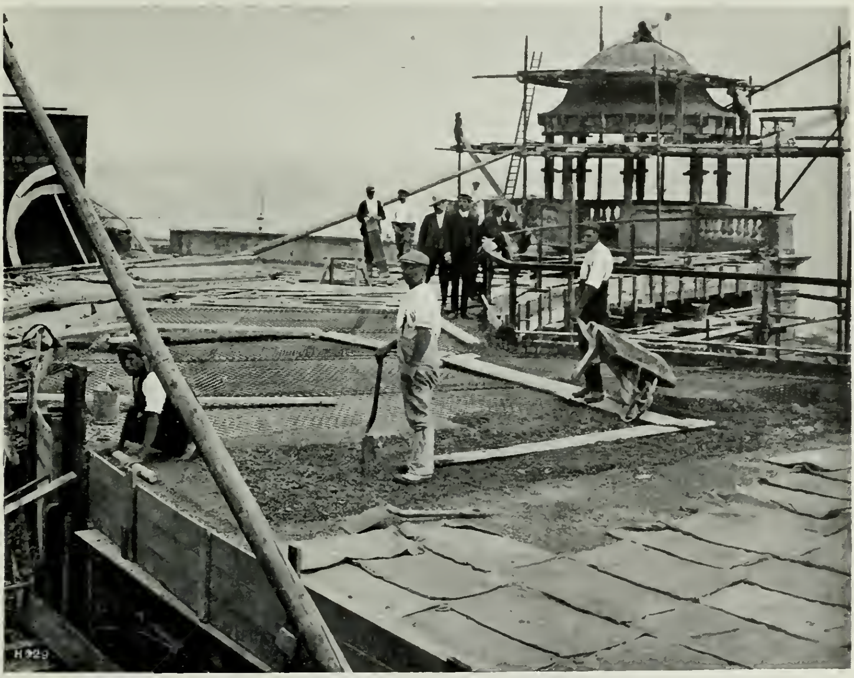


View of finished buildings.
REINFORCED CONCRETE COLONNADE AT BEXHILL.

REINFORCED CONCRETE COLONNADE AT BEXHILL.

The new Colonnade at Bexhill, which was opened recently by Lord Brassey, forms a very pleasing additional feature to the sea front. It is semi-circular in shape, having a diameter of about 140 ft., and has a bandstand situated at its centre.

The Colonnade throughout is of structural steelwork and Expanded-Steel concrete construction. The roof, which forms a circular promenade, is about 200 ft. long and 18 ft. wide; it is supported by steel columns and joists, and is constructed of concrete, about 6 in. in thickness, reinforced with No. 10 3-in. diamond-mesh Expanded Steel. The whole of the structural steelwork is encased in Expanded-Steel concrete, finished to resemble Bath stone.



View showing Expanded Metal being laid.
REINFORCED CONCRETE COLONNADE AT BEXHILL.

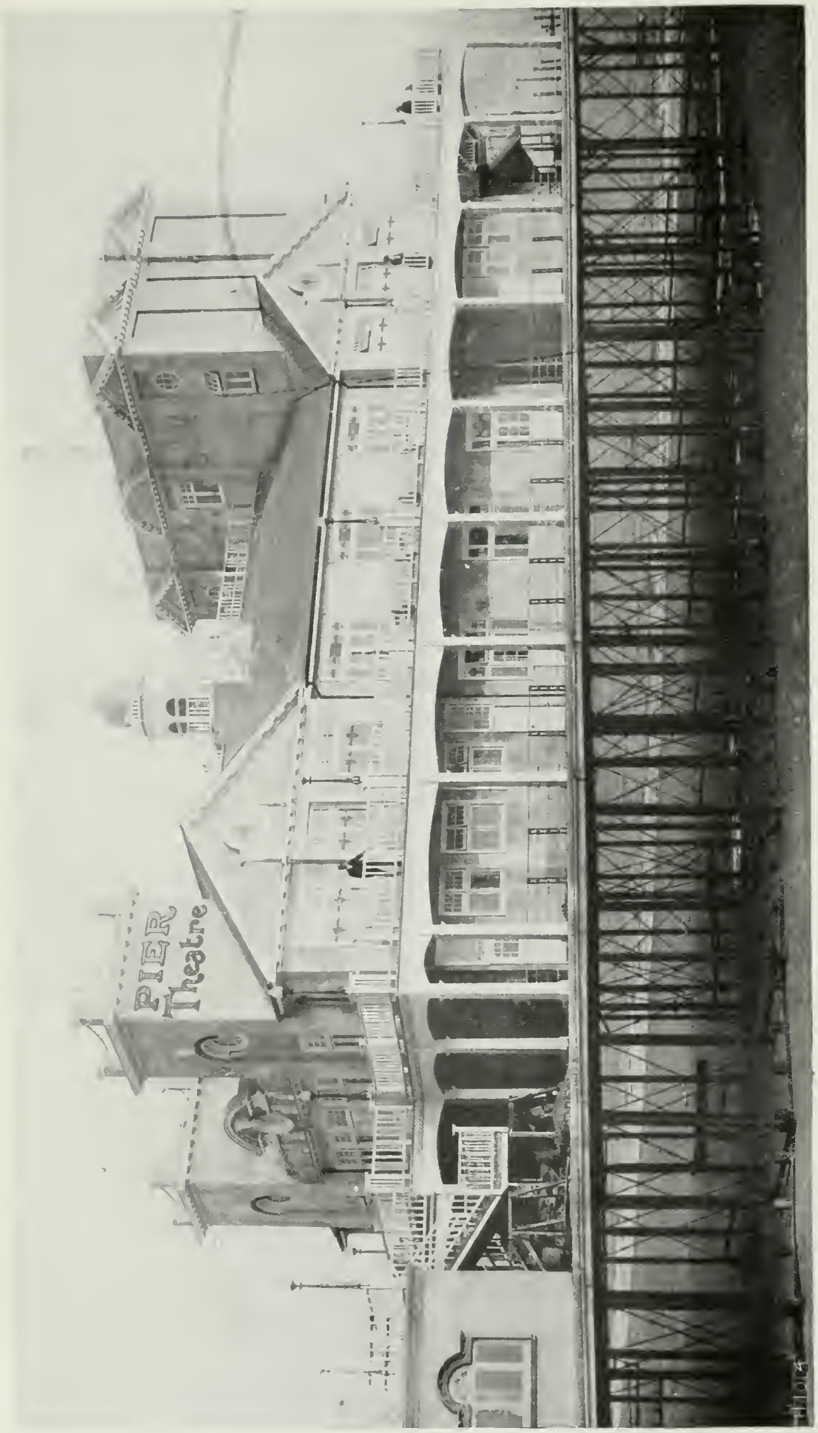
At each end of the Colonnade is situated a dome-roofed pavilion, while beyond them covered shelters with open terraces above have been constructed. The bandstand and pavilions are circular in shape, the ornamental roofs being supported by steel columns encased in Expanded-Steel concrete.

Councillor J. B. Wall, of Bexhill, was responsible for the design, which was presented by him to the Borough. The work was carried out under the supervision of Mr. G. Ball, A.M.Inst.C.E., the Borough Surveyor.

The General Iron Foundry Co., Ltd., of London, were contractors for the work, and the Expanded Steel used was supplied to them by the Expanded Metal Co., Ltd., of London and West Hartlepool.

REINFORCED CONCRETE WORK IN THE NEW PIER AND PAVILION AT BOGNOR, SUSSEX.

The accompanying illustrations show the new Pier and Pavilion, completed last summer, for the Bognor Pier Co., Ltd., to the designs and under the

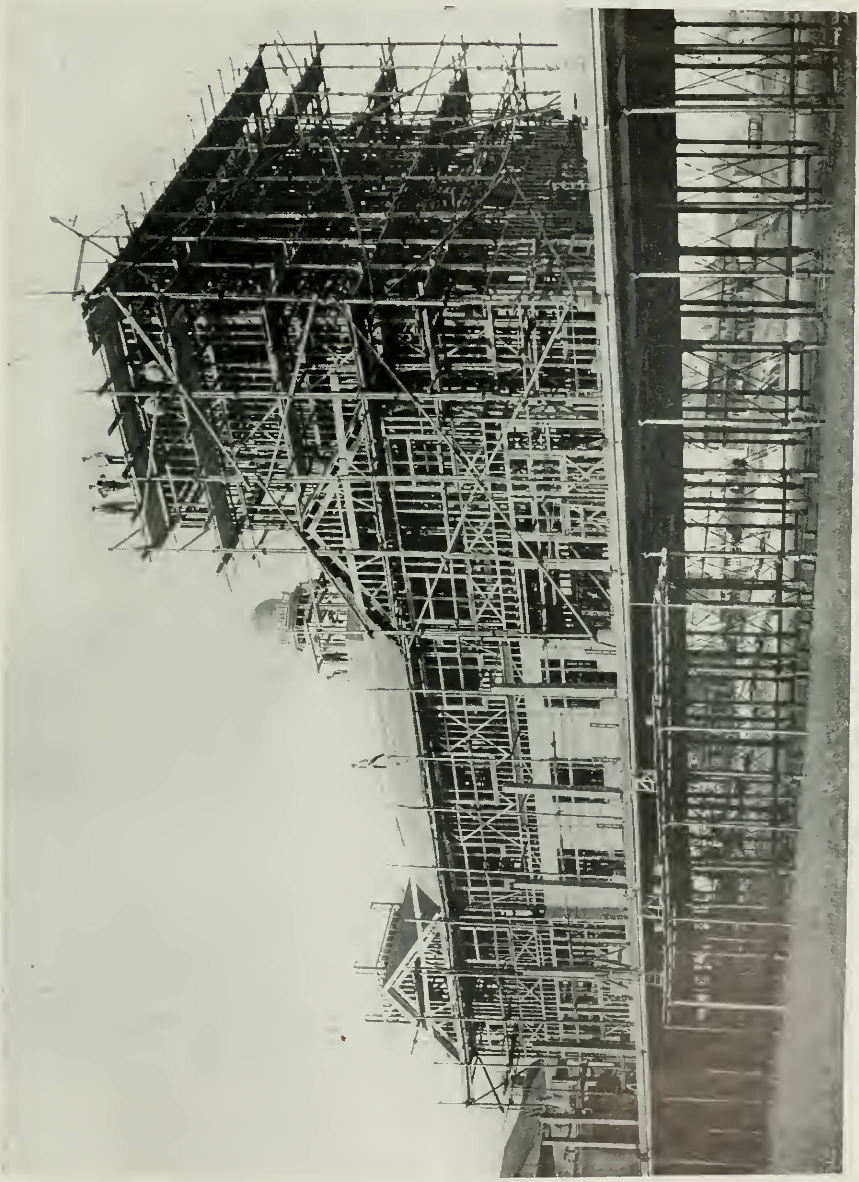


View of Pavilion Completed.

Reinforced Concrete Work in the New Pier and Pavilion, Bognor.

supervision of their architect, Mr. G. E. Smith, of Victoria Road North, Southsea, Portsmouth.

A pier had been in existence for many years, and, owing to the increasing popularity of Bognor as a seaside resort, it was decided to reconstruct and extend the old pier and to add a pavilion. For that reason the sub-structure was strengthened by the addition of screw piles and bracings, and steel and reinforced concrete beams for the decking, the old timber decking being removed and replaced by reinforced concrete decking.



Reinforced Concrete Work in the New Pier and Pavilion, Bognor.

View in Course of Construction.

The beams were reinforced with Expanded-Steel bars, and the decking with Expanded-Steel Sheets, the decking generally being 6½ in. in thickness at spans varying from 15 ft. 6 in. downwards. The decking was built on the continuous type, with one layer of Expanded Steel throughout near its bottom face, and tension strips of Expanded Steel, varying in length according to the span of the slabs, near the top face of the concrete, over intermediate supports.

The decking was tested with an evenly distributed superimposed load of 2 cwts. of shingle per foot super, the test proving to be eminently satisfactory.

The pavilion building is steel framed throughout, the walls being built by means of wood studs fixed 12 in. apart between the steel horizontals and uprights, and covered on both sides with Expanded-Metal lathing and cement plaster, in which way light hollow walling was obtained. The flooring and stairs throughout the building are of Expanded-Steel concrete.

Our illustration on page 103 shows the steel and timber skeleton framework practically completed, while that on page 102 shows the building completed, with its overcoating of Expanded-Metal lathing and cement plaster.

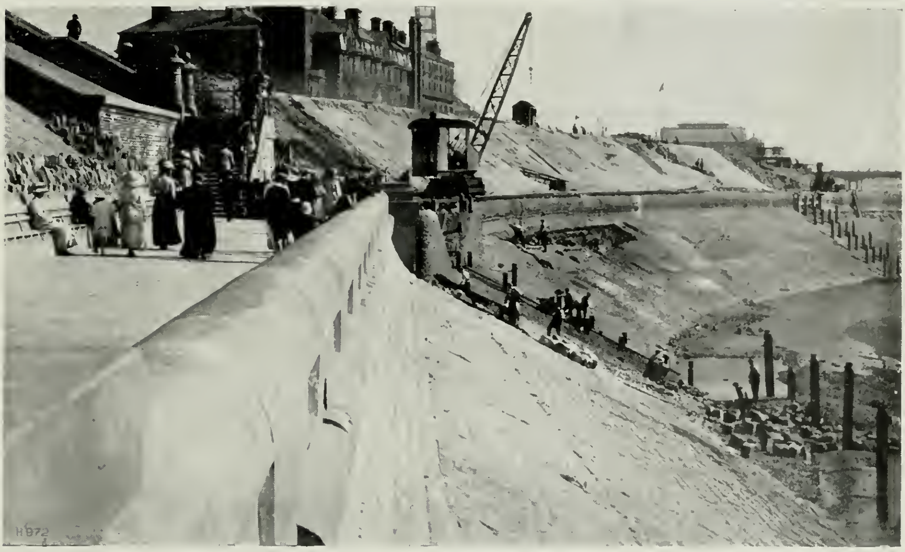
The details of the reinforced concrete work were prepared, under the direction of Mr. G. E. Smith, the architect, by the Expanded Metal Co., Ltd., of London and West Hartlepool, who supplied the Expanded-Steel reinforcements for the concrete work and the Expanded-Metal lathing for the plaster work.

View of Seats in Course of Construction.
New Sea Wall and Promenade, Blackpool.

NEW SEA WALL AND PROMENADE, BLACKPOOL.

The completion of the new Promenade extension between the North Pier and Cocker Street, Blackpool, concludes the large undertaking, begun eight years ago, of widening the Promenade between the South Shore and the North Pier. The work included the provision of esplanades and a substantial sea wall for a length of about three miles.

The sea wall varies in height between 26 ft. and 37 ft., and is 11 ft. thick at the base. At the toe of the apron, which has a slope of 1 to 1½, ring piles are driven 12 ft. apart, to which the permanent walling and sheeting piles are secured. The apron is formed of consolidated sand covered with a layer of concrete 2 ft. 6 in. thick, and faced with basalt grouted and bedded in cement mortar. No reinforcement was used in the sea-wall, as solidity and dead-weight are both essential in such an exposed position. The amount of Portland cement concrete used was 36,000 cubic yards.



View showing Closing-in of Apron.

New Sea Wall and Promenade, Blackpool.

The whole of the work was carried out departmentally by the Corporation, from plans prepared by the Borough Engineer, Mr. J. S. Brodie, M.Inst.C.E., assisted by Mr. H. Banks, who acted as Resident Engineer.

PROBLEMS IN THE THEORY OF CONSTRUCTION

CONCRETE BEAMS WITH DOUBLE REINFORCEMENT

By EWART S. ANDREWS, B.Sc.Eng. (Lond.).

The following article on "Concrete Beams with Double Reinforcement" is the first a series dealing with Problems in the Theory of Construction, by Mr. Ewart S. Andrews, which we intend to publish, and which should prove of great benefit to many of our readers. —ED.

ALTHOUGH the use of compression reinforcement for concrete beams is not economical, it is sometimes necessary, and the problem arises as to what is the strength of a beam in which the compression reinforcement is as much, if not more, than the tensile reinforcement.

If the tensile reinforcement is rather large, say 2 per cent., and sufficient compression reinforcement be added in order to develop the full tensile strength of the tension reinforcement, then, on the ordinary theory employed for calculating the strength of reinforced concrete beams, considerably more steel will be necessary on the compression side than on the tension side; this seems to be a very wasteful procedure which points to the desirability of employing a different method of calculation for these cases.

We shall get an interesting comparison if we consider the stages through which the stressing of a reinforced concrete beam goes in the case where a rectangular beam has, say, 2 per cent. reinforcement in both tension and compression.

We will take the following notation :

d = depth of beam in inches to centre of tension reinforcement.
n = depth of neutral axis in inches from top of beam.
m = modular ratio of steel to concrete (15).
b = breadth of beam in inches.
A = area of reinforcement = $\cdot 02bd$.

In order to be able to work in general terms we will take the centre of reinforcement at a distance $\frac{d}{8}$ from each edge.

FIRST STAGE—ALLOWING FOR TENSION IN CONCRETE.

Until we have exceeded the safe tensile stress in the concrete, the bending will take place according to the ordinary theory of bending, assuming the reinforcement replaced by m times its area of concrete. *Fig.* 1 shows the equivalent section for this stage. Then the moment of inertia

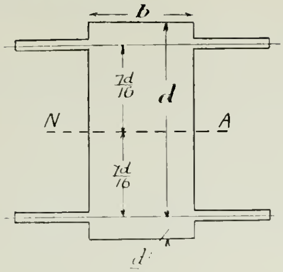


FIG. 1.

about neutral axis of the equivalent section, allowing for the hole in the concrete made by the reinforcement :

$$
\begin{aligned}
&= I = \frac{b}{12} \times \left(\frac{9d}{8}\right)^3 + 2\,(m-1)\,A\left(\frac{7d}{16}\right)^2 \\
&= {\cdot}1186bd^3 + {\cdot}1072bd^3 \\
&= {\cdot}2258bd^3
\end{aligned}
$$

$$\therefore \text{ Modulus of section} = \frac{I}{\frac{9d}{16}} = {\cdot}3613bd^2$$

This formula is, of course, very little used, because little attention is paid to the tension in the concrete.

SECOND STAGE—NEGLECTING TENSION IN CONCRETE AND ALLOWING FOR COMPRESSION IN CONCRETE.

When the safe tensile stress in the concrete has been passed, then the tensile resistance of the concrete is neglected altogether, this being the assumption upon which reinforced concrete beams are usually designed.

In this case we have first to find the position of the neutral axis, *Fig.* 1 showing the equivalent section.

This is done by equating the first moments of the equivalent areas about the neutral axis.

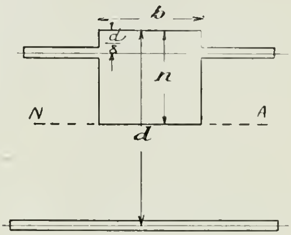


FIG. 2.

This gives: $$mA\,(d-n) = bn\,.\,\frac{n}{2} + (m-1)\,A\left(n - \frac{d}{8}\right)$$

or $$15 \times {\cdot}02bd\,(d-n) = \frac{bn}{2} + 14 \times {\cdot}02bd\left(n - \frac{d}{8}\right)$$

i.e. $${\cdot}3d\,(d-n) = \frac{n^2}{2} + {\cdot}28d\left(n - \frac{d}{8}\right)$$

$$n^2 + 1{\cdot}16nd - {\cdot}67d^2 = 0$$

The real solution of this quadratic equation gives :

$$
\begin{aligned}
n &= d\left\{\frac{-1{\cdot}16 + \sqrt{1{\cdot}16^2 + 2{\cdot}78}}{2}\right\} \\
&= {\cdot}435d
\end{aligned}
$$

Equivalent moment of inertia about neutral axis :

$$
\begin{aligned}
&= I = \frac{bn^3}{3} + (m-1)^* A\left(n - \frac{d}{8}\right)^2 + mA\,(d-n)^2 \\
&= \frac{{\cdot}435^3 bd^3}{3} + 14 \times {\cdot}02 \times {\cdot}31^2 bd^3 + 15 \times {\cdot}02 \times {\cdot}565^2 bd^3 \\
&= {\cdot}0274bd^3 + {\cdot}0269bd^3 + {\cdot}0957bd^3 \\
&= {\cdot}15bd^3
\end{aligned}
$$

∴ Modulus of section, taking compression in concrete :

$$S_m = \frac{I}{n} = \frac{{\cdot}15bd^3}{{\cdot}435d} = {\cdot}3448bd^2$$

Taking 600 lbs. per sq. in. as the safe stress in compression in the concrete, we shall have : Safe bending moment $= B = 600\,S_m = 206{\cdot}9bd^2$.

---

* $(m-1)$ *to allow for the hole made in the concrete by the reinforcement. This is a refinement which but few writers adopt, but it makes very little additional complication.*

THIRD STAGE NEGLECTING CONCRETE ALTOGETHER.

If we pay no attention to the concrete at all, and consider the resisting couple as given only by the stresses in the reinforcement, we shall get modulus of section = area of each reinforcement × distance between them

$$= {}^{\cdot}02bd \times \frac{7d}{8}$$

$$= S_m = {}^{\cdot}0175bd^2$$

$\therefore$ Safe bending moment $= B = S_m \times$ safe stress in reinforcement $= 12000S_m$, taking 12000 lbs. per sq. in. as the safe stress per sq. in. for steel in compression.

$\therefore$ $B = 12000 \times {}^{\cdot}0175 = 210bd^2$

This is a little more than the safe bending moment for the second stage, and so it would be adopted.

In order to get a good comparison of the three stages we will draw a diagram showing the variation of bending moment as the compressive stress in the steel increases.

Taking the breaking stress of concrete in tension as 240 lbs. per sq. in., the stress in compression in the steel will then be equal to

$$\frac{15 \times 240 \times \frac{7d}{16}}{\frac{9d}{16}} = 2800 \text{ lbs. per sq. in.}$$

because the stress in the steel is m = 15 times the stress in the concrete at the same depth, and the stress in the concrete is proportional to the distance from the neutral axis.

Then bending moment = stress × modulus

$$= B = 240 \times {}^{\cdot}3613bd^2$$

$$= 86{}^{\cdot}71\,bd^2$$

Now take the next stage. As we have previously shown, a bending moment of $206{}^{\cdot}9bd^2$ is obtained when the compressive stress of 600 lbs. per sq. in. is reached in the concrete. The stress in the compression reinforcement is then equal to

$$\frac{15 \times 600 \times {}^{\cdot}31d}{{}^{\cdot}435d} = 6{,}413 \text{ lbs. per sq. in.}$$

Again, from the third stage we get, as shown before, a bending moment equal to $210bd^2$ when the compressive stress in the steel reinforcement is equal to 12,000 lbs. per sq. in.

These results are shown diagrammatically in *Fig.* 3, which shows the variation in bending moment as the compressive stress in the steel increases.

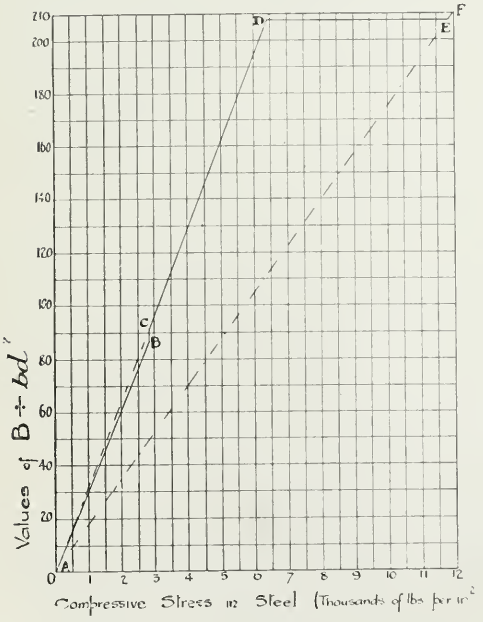


FIG. 3.

AB represents the first stage; at the point *B* the concrete begins to fail on the tension side, and the bending moment rises to *C* and then increases to the value *D*, *CD* representing the second stage. *AF* represents the increase in bending moment according to the third stage, the portion *EF* of this being the useful one, *DE* being horizontal. If, of course, the line *AF* lies entirely below *DE* the third stage has not to be considered.

The amount of reinforcement determines whether this third stage has to be considered. This is shown in *Fig.* 4.

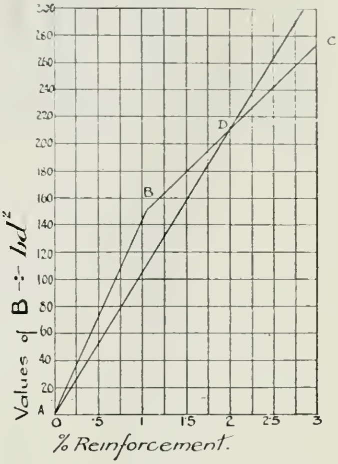


FIG. 4.

The curve *ABC* gives the values of $\frac{B}{bd^2}$ for various percentages of reinforcement from a consideration of the compression stress in the concrete, but neglecting the tension stress in the concrete, this being what we have referred to as the second stage.

It will be noted that this curve has two distinct portions. This is because for small percentages of reinforcement the limiting stress in the tension reinforcement is reached before the limiting compressive stress in the concrete is attained, so that in finding the safe bending moment we have to consider not the concrete but the steel. With the same notation as before we should have for such percentages :

$$\text{Modulus of section considering steel} = \frac{I}{(d-n)}$$

$$\therefore \text{ Safe B.M.} = \frac{16000I}{15(d-n)},$$ taking 16,000 lbs. per sq. in. as the safe stress in tension reinforcement.

The portion *AB* therefore represents the period over which the tensile limit is reached in the tension reinforcement, while the portion *BDC* represents the period over which the compression stress in the concrete is the determining factor. The line *ADE* shows the values of $\frac{B}{bd^2}$ for various percentages of reinforcement from a consideration of the compression stress in the steel and neglecting the resistance afforded by the concrete, this being what we have referred to as the third stage. From this diagram it is clear that for reinforcements up to 2 per cent. it is economical to design by the ordinary formulæ based upon the compressive stress in the concrete, but that above this it is more economical to design from the consideration of the compressive stress in the steel.

We have limited our consideration to the case in which the area of reinforcement in tension and compression are equal. Similar calculations can be made for other cases where the reinforcements are other than this.

The writer has not seen the results of tests of beams with double reinforcement in which the question has been investigated as to whether the strength goes according to the third stage, when the area of reinforcement is large.

Some further experimental investigation of the strength of reinforced concrete beams will be very welcome, especially if carried out with the care and thoroughness which has characterised the work done by Professor Talbot, for instance, at Illinois University. Theoretical treatments are always interesting, but they lose considerably in value if not justified by carefully conducted experiment.

"EVERY DAY USES OF PORTLAND CEMENT."

The second edition of this extremely useful handbook on the Uses of Portland Cement has just been published, and should prove of inestimable value to all users of that material.—ED.

The Associated Portland Cement Manufacturers (1900), Ltd., were well advised when they published their popular handbook entitled "Every Day Uses of Portland Cement." It was a pioneer work of the first order and did an immeasurable amount of good, both at home and in the Colonies, in popularising the subject.

We constantly come across the book in all manner of unlikely places. We have seen it in an estate foreman's home, in a farmer's parlour, on the desk of a great Government engineer; we have come across copies in an Egyptian works department and on a sapper's desk in Gibraltar. We have had endless inquiries for the book, and since it has been out of print we are credibly informed that the waiting list for the second edition has mounted to several hundred. In other words, the book has been a great success, and its conception is a matter of congratulation.

A second edition has now been published, and we welcome its appearance. It contains some additional matter, some corrections, a number of fresh illustrations, and a most useful index. There is no great change in the character of the volume. The original scheme has been adhered to, and we can commend the volume to our readers.

We have, however, one somewhat emphatic comment to make. The book is not sufficiently British in its examples for 1912. When originally produced it was no doubt very difficult to find British examples of the work to which the Associated Portland Cement Manufacturers wished to call attention, but to-day a motor-car ride within a radius of 100 miles around the Metropolis should suffice to fill the book with useful examples, as Portland cement is now freely used both for popular and highly technical purposes all over the British Isles. Again, if searching for examples afar, why not turn to our Colonies? We think the Company have missed the opportunity of showing the splendid effects of their pioneer work at home and in the Colonies, where their propaganda has borne excellent fruit.

We see that a supplement may be issued. We would strongly urge it to be devoted to British and Colonial examples, and emphasise the progress in the use of Portland Cement being made throughout the Empire.

"Every Day Uses of Portland Cement." Published by the Associated Portland Cement Manufacturers (1900) Ltd., Portland House, Lloyd's Avenue, E.C. Second Edition. Paper covers, 1s. 6d. net; stiff covers, 2s. 6d. net.

EVERY DAY USES OF PORTLAND CEMENT.

The second edition is greatly enlarged, containing 340 pages, and is profusely illustrated. It gives much useful information and is absolutely essential on the desk of all those interested in building construction of every description.

From the following chapter headings the wide scope of the book will be seen :—

Portland Cement, dealing with the difference between Portland cement and natural cement; also slag cement, etc.

Aggregates, dealing with the Selection and Proportioning of Aggregates—Percentage of Voids—Mixing—Cracks in Concrete—Estimating the Cost of Concrete Work.

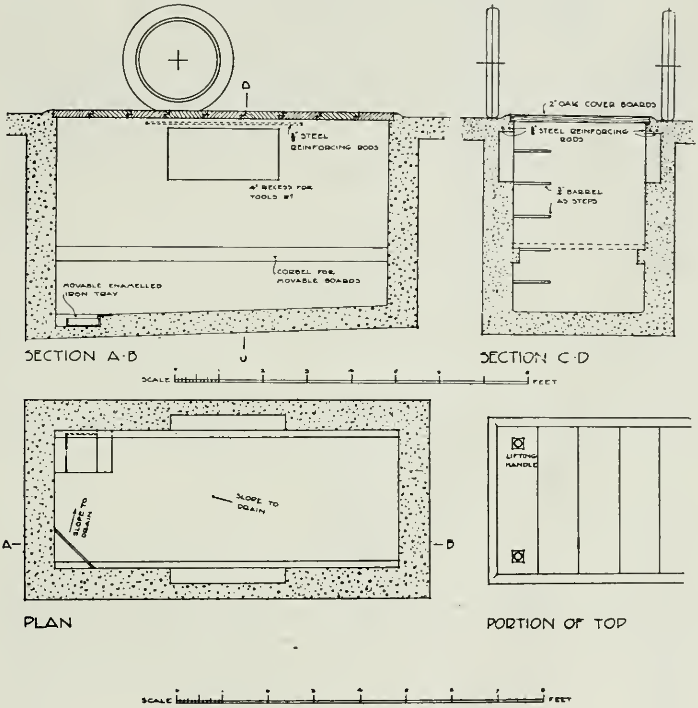


Fig. 1. Plan and Sections of a Concrete Motor Pit.

Workmanship.—This includes Construction of Forms—Centering—Putting Concrete in Place—Placing Concrete in Water—Surface Finish—Waterproofing—Cement Mortar—Tools used for making Concrete, etc.

Concrete Blocks and Moulded Concrete.—This chapter also gives machines for block-making—Concrete Bricks and Roofing Tiles, etc.

Reinforced Concrete.—This devotes a great deal of space to calculations for reinforced concrete work.

Memoranda for Concrete Users.

The Uses of Concrete.—The remainder of the book, some 200 pages in all, is devoted to examples of the numerous uses of concrete.

We propose to quote a few examples of new uses illustrated in the book, but are in some difficulty, as so many of those reproduced have previously appeared in our columns—in fact, are largely from engravings of our own. The following, however, should be new to most of our readers, and will serve as good examples of useful illustrations to be found in the book :—

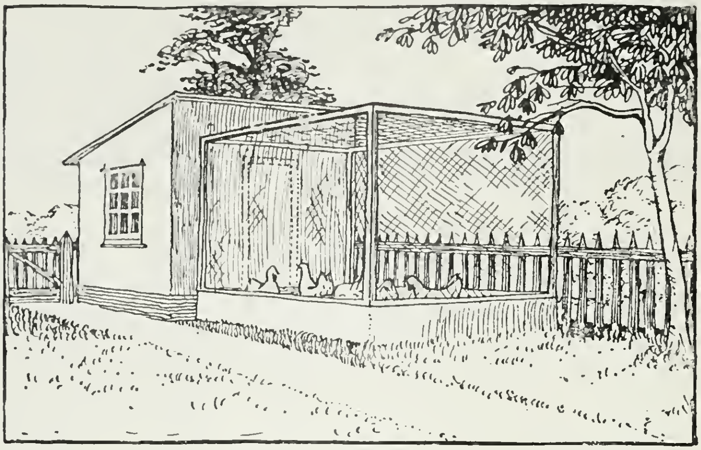


Fig. 2. Concrete Chicken House.

Fig. 1 gives details of the construction of a motor pit in concrete. Satisfactory dimensions for such inspection pits, which are very necessary for the proper overhauling of the interior of cars, have been found to be the following: 6 ft. by 3 ft. by 4 ft. 6 in., but these can, of course, be varied according to circumstances. A shelf or recess on either side of the pit is an advantage ensuring easy access to the tools, etc. A suitable cover can be made of 2-in. boards, to each of which a sunk lifter should be provided.

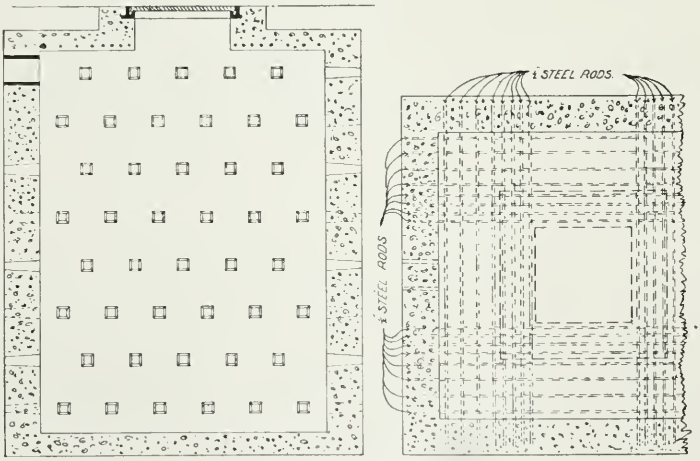


Fig. 3. A Concrete Cesspool.

Fig. 2 shows a chicken house made of concrete. This material is particularly advantageous for such a purpose, as it resists the incursions of rats, weasels, and other vermin. Such a chicken house may be built with solid concrete walls 5 in. thick. If the house is more than 8 ft. wide, the roof, sloping one way only might be made of a 4-in. concrete slab with reinforcement of steel rods or heavy meshwork. The slope should be 1 in. in every foot, and the surface should be well trowelled to make it waterproof. If the roof is more than 8 ft. span, concrete beams or rafters should be adopted with a top slab; or a pitched roof may be used, either of concrete or other material. Concrete shelves and water basins may also be adopted. The use of coke breeze or clinker is advised for the aggregate in constructing such a building, as it gives sufficient porosity to the concrete to prevent moisture condensing on the inside of the walls in cold weather. The walls should be coated with a Portland cement wash as soon as the forms are removed.

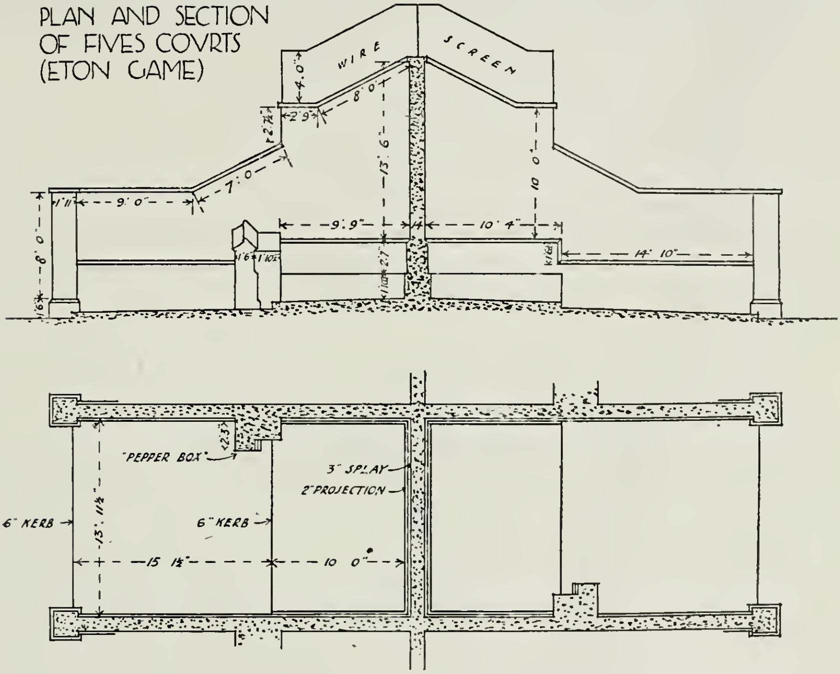


Fig. 4. Plan and Section of Fives Court.

Fig. 3 shows the construction of a cesspool for use where there are no public sewers.

Another use to which reinforced concrete can be advantageously applied is in the construction of Fives and Racquet Courts, and *Fig.* 4 shows a Fives Court on the Eton model. Both these require high walls and a paved floor. The walls can be built thin, panelled at the back—*i.e.*, stiffened by ribs, braces and buttresses. The reinforcement need only serve to resist stresses, and the effect of temperature changes which would be inclined to crack plain concrete. The surface finish of the walls should be trowelled with cement and sand.

SOME THERMAL PROPERTIES OF CONCRETE.

By CHARLES L. NORTON.
Professor of Heat-Measurement,
Massachusetts Institute of Technology, Boston, Mass.

During the last three years a series of experiments have been carried on in the Laboratory of Heat Measurements of the Massachusetts Institute of Technology, having for their object the study of those physical properties of Portland cement concrete which affect its value as a fire resistant material. While these researches are by no means complete, it is perhaps of interest at this time to discuss some of the results obtained.

It was proposed at the outset to make a study of the various physical properties of concrete over as wide a range of temperatures as possible, and among those selected were the following:

1. Coefficient of linear expansion.
2. Diminution of mechanical strength after heating.
3. Specific heat.
4. Coefficient of thermal conductivity.

1. COEFFICIENT OF LINEAR EXPANSION.

The measurements of the coefficient of linear expansion are practically complete, but the uncertainty attaching to some of the high temperature values, though slight, makes it seem wise not to report in detail until one or two check measurements can be made. This uncertainty arises from the great difficulty which attends the measurement of the temperature of the interior of a large block of cement concrete so as to be sure of its absolute uniformity at high temperatures. Thermal junctions embedded in the block at distances of 3 in. from one another are not sufficiently close to give the desired precision.

The method adopted for the measurements of the elongation caused by heating was the common so-called telescope method. The specimens in the shape of 6-in. or 10-in. cubes were slowly heated in a double gas muffle or an electric resistance furnace. The temperature of the furnace and of a number of points in the concrete was taken by means of platinum-rhodium couples.

Near the furnace were mounted two telescopes, *Fig.* 1, which might be sighted through holes in the furnace wall upon reference points on the surface of the block. At low temperatures an arc light and a system of mirrors were used to furnish adequate illumination. One of the telescopes was provided with a micrometer eyepiece by means of which a movement of the reference mark of ·0001 in. could be measured.

The values obtained at low temperatures agree very well with the commonly accepted value of ·0000055 for the elongation per unit of length per degree Fahrenheit. Apparently this value increases slightly up to 575° F. Above this point the coefficient becomes smaller; at 1,500° F. the coefficient becomes zero, and above this point slightly negative. On cooling the blocks which had been heated to 1,500°, they did not return to their original dimension, their permanent elongation being about 75 per cent. of their maximum elongation. On a second heating the specimens all returned to the same dimensions, there being no sensible permanent elongation resulting from the second heating.

All of the specimens tested for expansion were of stone concrete, of the proportions 1 : 2 : 5. The stone was clean, the sand sharp, the cement of good quality, and every precaution was taken to secure a concrete of the first order. The specimens weighed on the average 150 lb. per cu. ft.

2. DIMINUTION OF MECHANICAL STRENGTH AFTER HEATING.

In order to study the effect of high temperatures upon the compressive strength of concrete several scores of 6-in. cubes were made and allowed to set for 90 days or slightly longer. These blocks were then heated for different lengths of time at different temperatures in a gas furnace similar to that used for the expansion experiments.

The cubes which were not heated showed an average compressive strength of 2,700 lb. per sq. in. Those which were heated to 932° for two hours lost only 15 per cent. of their original strength, and when this temperature was prolonged for four hours the loss was 43 per cent. If the blocks were heated to 1,500° for two hours the loss of compressive strength was 38 per cent., and at the end of four hours the loss was 66 per cent. The most important information secured from these tests was to the effect that even after exposure to a fire in which the concrete block, small in size, was subjected to flames at a temperature of 1,500° for four hours, it was still able to carry twice its normal safe load in compression. The larger beams with their great masses are not often exposed for so long a time to so great a hazard.

It should be noted, however, that there was a considerably greater deformation under load of the heated blocks than of those not heated.

A large number of small beams were next made, some with and some without reinforcement; most of these were either 6 × 6 × 48 in. or 8 × 8 × 48 in. The specimens which were reinforced contained four ½-in. round steel rods situated near the corners, equi-distant from the two faces of the beam. In some the distance from the reinforcement to the face of the beam was 1 in. and in others 1½ in. A few beams had a 2-in. protection to the reinforcement. Not many of these specimens have been tested as yet, but, from a few, we are able to get a general idea of what happens to beams of this type in case of exposure to fire.

Three beams for instance, each 6 × 6 × 48 in., in which the reinforcing rods were 1 in. from the face of the beams, were broken by centre load, the first beam not having been heated at all, the second heated for one hour in a fire that fused the surface of the concrete, and the third being similarly heated for two hours. The beam which was not heated broke under a load of 5,700 lb.; the second, heated for one hour, broke at 2,750 lb., while the third, heated for two hours, broke at 1,950 lb. I consider this a most remarkable showing under severe conditions. It should be borne in mind that these small beams were so slow in cooling down that they were in effect heated much longer than the time mentioned. The flames, moreover, surrounded the beams on all sides.

Several similar tests were made with concrete beams of the same dimensions, 6 × 6 × 48 in., with no reinforcement. These were heated for two and four hours at 1,700° F. After this exposure all of the beams were very frail, not one of them showed any cracking or spalling, however. It was not considered necessary to break any of these beams, as it was apparent that the concrete on the tension side of the beam was much weakened. Some of the more recent experiments made on beams 8 × 8 × 48 in. show that these larger beams are much less weakened, proportionally, as would be expected to be the case.

A series of similar beams were next made up of cinder concrete, the proportions of the mixture being 1 : 2 : 5. A portion of these were mixed with clean cinders, which showed upon analysis but little carbon; a second part was mixed with cinders to which 10 per cent. of fine bituminous coal had been added, and the other beams were mixed with cinder, to which had been added 25 per cent. of fine coal. The 25 per cent. mixture can be disposed of in a word. When once thoroughly heated it burned until it fell to pieces. With the 10 per cent. mixture, however, no such action occurred; there was no indication that the concrete would support its own combustion even for a short time. It was apparent that the 10 per cent. mixture was not so good a fire-resistive material as that which contained no added carbon. From the few specimens containing less than 10 per cent. which have been examined up to the present, it seems probable that the safe limit is close to 5 per cent. More information is now being secured on this point.

3. SPECIFIC HEAT.

The study of the specific heat of concrete was made by the ordinary calorimeter method, the "method of mixtures" of Regnault. Specimens of the concrete, usually fragments of the larger test pieces, were heated slowly in an electric resistance furnace to the desired temperature and then plunged into the calorimeter. The weight of the water and its rise in temperature give the amount of heat given off by the body in cooling.

Extraordinary precautions were taken in getting the exact average temperature of the specimen in the furnace, and to insure its rapid transfer to the calorimeter. In most of the experiments a double calorimeter was used so that the specimen did not come in contact with the water of the calorimeter, so that any evolution of heat by hydration of the cement was avoided. Table I. gives the specific heat of concrete.

TABLE I.—SPECIFIC HEAT.

Temperature. Degrees Fahrenheit.	Stone Concrete 1 : 2 : 5	Stone Concrete 1 : 2 : 4	Cinder Concrete 1 : 2 : 4
72 to 212	.156	.154	.——
72 to 372	.192	.190	.180
72 to 1172	.201	.210	.206
72 to 1472	.219	.214	.218
72 to 2172	.——	.——	.——

4. COEFFICIENT OF THERMAL CONDUCTIVITY.

The measurements of thermal conductivity were made by a number of methods and have taken far more time and energy than all the others put together. The thermal conductivity is that property which determines how rapidly heat will travel through a substance and how rapidly therefore objects beyond will be heated by transmission. The conductivity becomes of prime importance in all questions of protection of the metal in reinforced concrete buildings. There is very little data to be found as to this important property of any of the common materials of engineering and such data as are to be found are not at all concordant. As to the conductivity of concrete or its variation with temperature and with composition, practically nothing has been known.

The methods adopted for the measurements will be here described in outline only. The formula showing the relation of the temperature upon the two sides of a plate to the amount of heat which would flow through it is as follows:

$$Q = \frac{K\,(t_1 - t_2)\,s\,A}{d}$$

or

$$K = \frac{Q\,d}{(t_1 - t_2 A\,s)}$$

where K = the coefficient of thermal conductivity dependent upon the nature of the material and its temperature.

Q = the quantity of heat flowing through the plate in the area measured.

A = the area.

t_1 = the temperature of the hotter side of the plate.

t_2 = the temperature of the cooler side of the plate.

d = the thickness of the plate.

s = time during which Q units flow through the area A.

The formula will be seen to be merely an expression of the following relations, that the flow of heat is proportional to the area, to the temperature, and the time, and that it is inversely proportional to the thickness.

After spending many months in developing other methods the electrical method used by the writer for the past fifteen years in studying the flow of heat through steam pipe coverings was adopted. The value Q of the heat flowing was determined by supplying the heat by means of the heating of a conductor carrying a current of electricity. By measuring the electrical energy supplied the quantity of heat developed may be known with great precision. Further, if this heat is passed through the plate under test and into a calorimeter on the far side, a check upon the value of Q may be had. For the determination of the temperature different thermal couples, resistance thermometers, and mercury thermometers were used, but thermal junctions made of thin strips of copper and nickel or of platinum and platinum-rhodium were generally found most serviceable.

The apparatus used for the lower temperatures consisted of a thin, electrically-heated plate, to the two sides and edges of which concrete could be applied. Outside of the concrete there were then placed heavy copper or brass plates which could be kept at a constant temperature by an internal circulation of water. Thermal junctions were placed at several points on each surface of each concrete plate. The electrical input was measured by calibrated Weston instruments, and calibrated thermal junctions gave the value of the temperature difference to the nearest one-one-hundredth of a degree. For the thickness, numerous measurements were made with a pair of flat-nosed calipers and averaged.

It was necessary to keep this apparatus running for several days before it could be balanced, that is, before the rate of flow of heat outward through the plates became constant and equal to the electrical input. The number of measurements has now reached many hundreds, and it seems apparent that the precision of the method depends on the accuracy with which the thickness of the specimens can be maintained constant. Otherwise the accuracy is better than 1 per cent. This apparatus has been named by my assistants the "flat plate tester," and perhaps it is well to so designate it here.

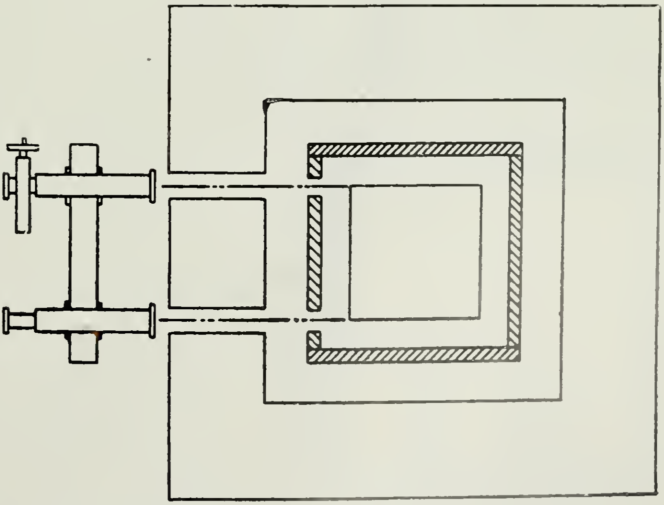


FIG. 1. DIAGRAM OF APPARATUS FOR MEASURING COEFFICIENT OF EXPANSION OF CONCRETE.

Later, in order to make tests on plates as thick as some of the walls in common use, another method was adopted. Cubical boxes 36 in. in outside dimension were built with walls of several thicknesses. Inside the boxes were placed electric heaters which served to raise the inside surface to a temperature above that of the surroundings. A sketch of the apparatus is shown in *Fig. 2.* A small fan served to keep the air in the box stirred to insure uniformity of temperature throughout. The boxes were tightly sealed. The power supplied to both heater and fan was measured as before. Mercury thermometers and thermal junctions, as well as a Callender recording resistance thermometer, were used to measure the difference in the temperatures inside and outside of the box. Data has been secured on scores of specimens and it is practically identical with the results obtained by the flat plate tester.

For the high temperatures a modification of the entire process was found necessary. The concrete to be tested was cast in the form of a cylinder on the outer surface of and concentric with a steel bar which could be heated to a high temperature by the passage of a heavy current. Outside of the cylinder of concrete was applied a closely-fitting "continuous" calorimeter. The temperatures of the bar and of the calorimeter were measured by thermal junctions, and the amount of water and its rise in temperature gave the value of *Q*.

In order to guard against the uncertainty of the temperature at the ends of the bar, the calorimeter was made so as to enclose only about one-half the length of the bar, the rest being covered by guard rings similar to the calorimeter but without provision for the measurement of the quantity of water.

TABLE II.—COEFFICIENT OF THERMAL CONDUCTIVITY OF CONCRETE.

Temperature of hot side of plate. Degrees.		Mixture.	Coefficient. Calories per 1 deg. C per sq. cm. per cm. per second.	Coefficient. B. T. U. per 1 deg. F. per sq. ft. per in. thick per 24 hours.
Centigrade.	Fahrenheit.			
35	95	Stone 1 : 2 : 5	.00216	150.
50	122	Stone 1 : 2 : 4 not tamped	.00110 to .00160	76. to 114.
50	95	Cinder 1 : 2 : 4	.00081	56.
200	392	Stone 1 : 2 : 4	.0021	146.
400	752	Stone 1 : 2 : 4	.0022	153.
500	932	Stone 1 : 2 : 4	.0023	160.
1000	1832	Stone 1 : 2 : 4	.0027	188.
1100	2012	Stone 1 : 2 : 4	.0029	202.

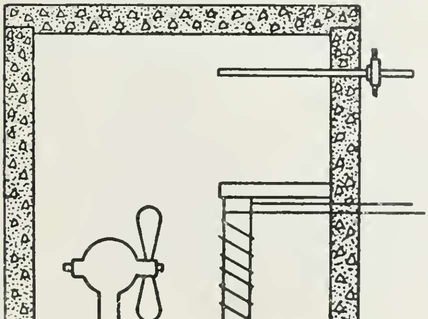


FIG. 2. ELECTRICALLY-HEATED CUBE FOR TESTING CONDUCTIVITY.

The heating of the bars required a considerable amount of special apparatus, since it was necessary to provide a current of upwards of 2,000 ampères for the high temperatures, and to be able to vary its amount to any desired value below that point. For this purpose there were installed three 15 k.w. transformers connected on the primary side with a three-phase 2,300 volt circuit. By means of divided secondaries and a rather elaborate arrangement of switches, the secondary voltage could be varied from 190 volts down to 55 volts. This secondary voltage was applied to the primary of a second step-down transformer, whose secondary was divided into twenty coils. By means of a switchboard the entire output of the transformer could be had at almost any desired low voltage. This enables us to heat bars insulated by materials of different composition and of different thicknesses to any desired temperature up to 2,800° F. With this arrangement both the steel and the concrete can easily be melted.

The results obtained are given in Table II. It is to be regretted that there is no uniformity of practice as to the units to be adopted in reporting the measure of effectiveness of insulators. While the physicist renders his report in calories per square centimetre, per centimetre thickness, per 1° Centigrade per second, the steam engineer confines his observations to B.T.U. per hour, per square foot, per inch of thickness, per 1° F., and the refrigerating engineer reports on the basis of a 24-hour time unit. I have even seen a report in terms of hogsheads of water raised to the boiling point, time not stated.

A brief comparison of these values with those for other materials may be interesting.

The specific heat of concrete is slightly less than that of either red brick or fire brick, hence the same amount of heat needed to raise the temperature of a pound of brick is about 10 per cent. more than for a pound of concrete. But the density of concrete is enough greater than that of brick to raise the heat capacity of a cubic foot of concrete above that of brick. The difference is not large, however.

While the expansion data is not complete, it seems clear that for a time after the beginning of exposure to fire, the concrete and its reinforcement will expand at much the same rate, but that the further expansion of the surface will not proceed at so rapid a rate. This will tend to reduce the stresses which the expansion of the heated surface would otherwise set up in the cooler interior. It is perhaps because of the failure of the concrete to return to its original dimensions that the small amount of surface cracking found after a fire is due.

The experiments made with coal and cinder mixtures indicate the necessity of added care in the selection of cinders for this purpose.

The table of thermal conductivities gives us data as to the rate at which heat will travel through concrete. It is interesting to note the great difference between the tamped and the untamped concretes made from stone. The one was as porous as possible, and the other as dense. One transmits nearly twice as much heat as the other. The cinder concrete, as is commonly believed, is much better as a heat insulator than the stone concrete, being nearly three times as effective as the denser stone concrete in retarding the flow of heat. It may be interesting to call attention to the heat insulation afforded by other materials. The best of the commercial articles used for this purpose is compressed cork, which is nearly 25 times as effective as stone concrete. Steel, on the other hand, transmits heat from 75 to 100 times as fast as the densest of the stone concrete.

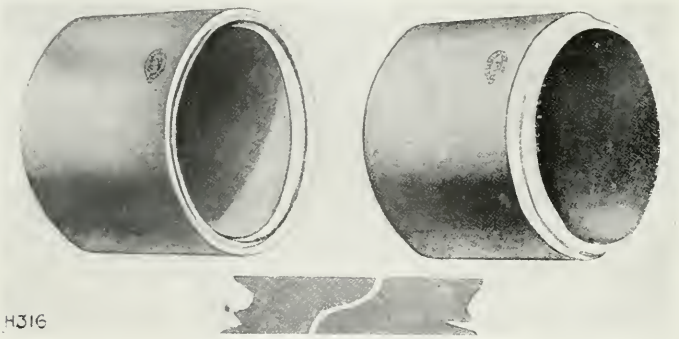
CONCRETE SEWER PIPES AT BRIDLINGTON.

By E. R. MATTHEWS, A.M.Inst.C.E., F.G.S., Borough Engineer.

The advantages of using concrete tubes as sewer pipes has been dealt with more than once in this journal, and we have given numerous examples of their increasing use for this purpose.—ED.

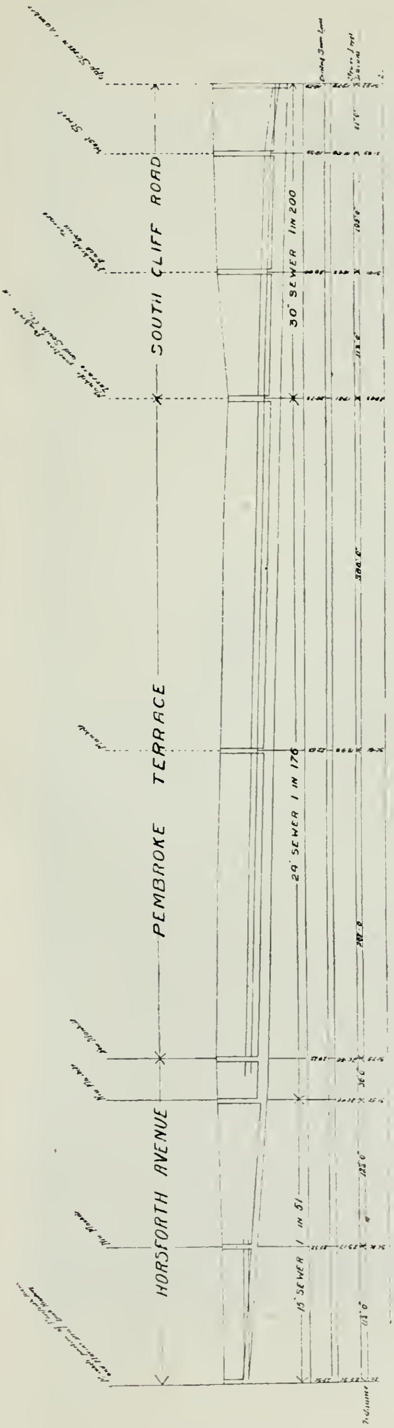
The advantages of using concrete tubes, if well matured, instead of the ordinary fireclay pipe in the construction of sewers have already been set out by the author in a previous issue of this journal. It is his intention in this short article to describe the use by him recently of granite-concrete tubes in constructing certain new sewers at Bridlington.

Remodelling of the Hilderthorpe Sewerage System.—Owing to the rapid development of the portion of the town on the south side of the harbour, known as the Hilderthorpe District, the author recommended the Town Council to allow him to prepare a scheme for the re-modelling of the main drainage system in this district. This he did, and the scheme, which was approved by the Local Government Board, included the taking up of some of the main sewers, which were not only altogether inadequate as regards their discharging capacity, but were also found to be very defective, many of the pipes that were bared for the purpose of inspection being found broken in several pieces. Scarcely one sound house connection was found to the old sewers when carrying out the work.



Circular Tubes for Sewer Pipes.

In the author's scheme he allowed for the substitution of 30-inch granite-concrete sewer tubes in place of the existing 18-inch sewer in South Cliff Road,



Concrete Sewer Pipes at Bridlington.

and 24-inch concrete tubes to take the place of the old 15-inch sewer in Pembroke Terrace. In both cases the new sewer was to be laid 4 ft. deeper than the old, so as to allow of the drainage of a district at present outside the Borough boundary, but which may in a year or two be annexed.

As concrete tubes were laid only in the two streets named, the remainder of the scheme for re-modelling will not be discussed.

The author will now briefly describe (*a*) the granite-concrete tubes used by him; (*b*) difficulties to be overcome in using tubes of this class in sewer construction.

(*a*) ***The Granite-Concrete Tubes Used.***—The author has no interest whatever in the tubes manufactured by any particular firm, and therefore is able to speak freely regarding the tubes used by him. These were supplied by Messrs. John Ellis & Sons, Ltd., of Leicester; the 30-in. were 2½ in. thick, the 24-in. 2 in. thick, and both were in 3 ft. lengths. They cost, delivered to Bridlington, 6s. 6½d. and 3s. 10½d. respectively per foot run, and if reinforced would have cost 6s. 11d. and 4s. 2d. per foot run respectively.

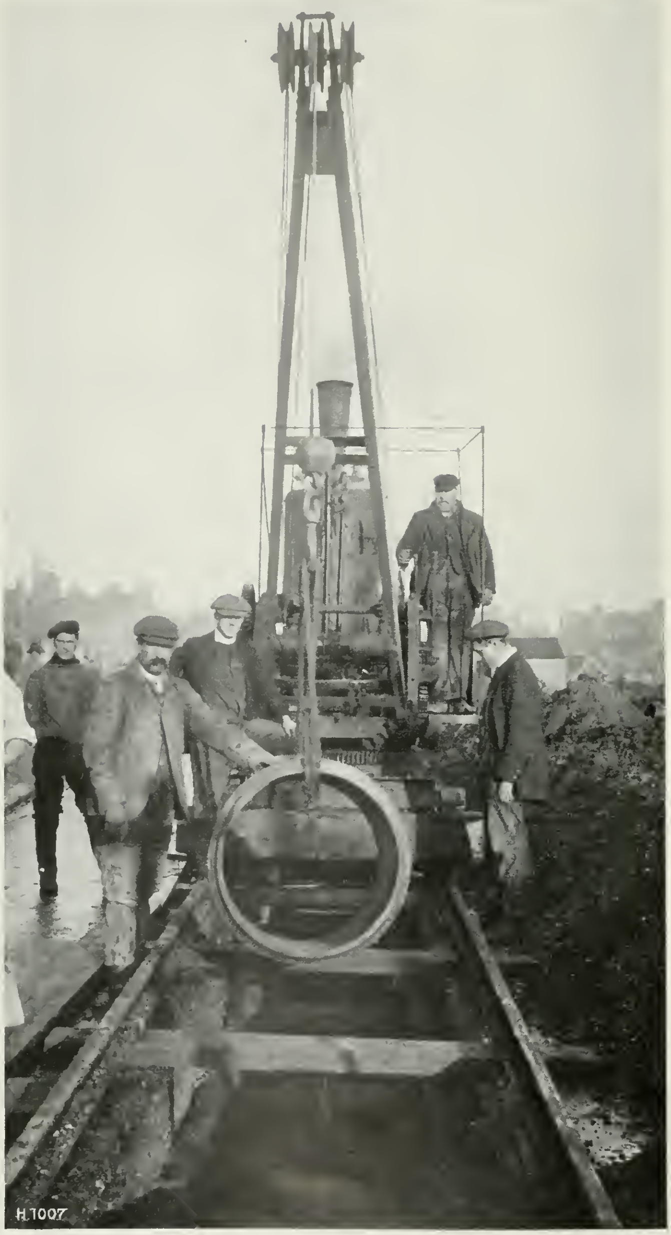
The tubes are manufactured as follows: Portland cement and Mountsorrel granite are used in the proportions of 1 to 3. The granite is crushed, washed, and passed through a ¼-in. mesh.

When reinforced, steel ribs are inserted; these are ¼-in. round steel wire rings, welded and worked into the material 9 in. apart.

8½ ft. of the 30-in. tubes equals in weight 8⅕ ft. 1 ton; and in the 24-in. tubes 13½ ft.

(*b*) ***Difficulties to be overcome in using tubes of this class in sewer con-***

struction. Splayed Joints.—The joints of these tubes being splayed, it requires the greatest care and patience to get a good joint. It will frequently happen that the tubes have to be turned round several times before one will fit well into the other, and it frequently occurs that to get the lower half of the pipe to fit well, the upper half has a wide joint.



The author considers that the joint might be very much improved. He would not recommend similar tubes to be laid in bad ground, unless a good bed of concrete was first put in. Most of the tubes laid by him were on a good clay bottom; some of the 24-in. tubes in Pembroke Terrace, however, were in a bad cutting, and had to be laid on a concrete bed.

Junctions.—It is better to cut these on the site; the exact angle at which it is required to connect the drain can then be ascertained more readily. The cutting of the tubes for the junction pipes should be done with the greatest care, and at least twenty-four hours should elapse between the making of the junction and the laying of the tube in position.

Handling of the Pipes.—The tubes being somewhat heavy, there is a temptation on the part of a contractor to handle them roughly. The author found that by using a 4-ton crane there was no fear whatever of breaking a pipe.

Plain Concrete** versus **Reinforced Tubes.—The firm supplying these tubes recommend that in a cutting not exceeding 20 ft. in depth plain concrete tubes should be used; at a greater depth than this they suggest reinforced tubes. The maximum depth of the South Cliff Road cutting was 16 ft., and of the Pembroke Terrace cutting 11 ft.

Notes in General.—Ninety-five lineal yards of the 30-in. tubes were laid, and two hundred and three yards of the 24-in.; the former at a gradient of 1 in 200, the latter at 1 in 176.

Pipes should be well matured.—This is the most important point in connection with the use of concrete tubes. They may be, and will be if well matured, most successful; or may, on the other hand, if not properly matured, be a failure. The firm supplying them should on no account send out tubes that fail in this respect. The engineer using them cannot be expected to test the tubes any more than he would test ordinary fireclay sewer pipes; he takes it for granted that the reputation of the firm sending out the tubes is at stake when these materials leave their works.

The contractor for these new sewers at Bridlington was Mr. Brumby Robinson, of Hull, and the works were executed under the able supervision of the Clerk of Works, Mr. W. Barugh, of Bridlington.

THE CONSISTENCY OF CONCRETE.

Interim Report of the Reinforced Concrete Practice Standing Committee of the Concrete Institute on the Consistency of Concrete.

The following report of the Reinforced Concrete Practice Standing Committee of the Concrete Institute on "The Consistency of Concrete" was read at a General Meeting on January 11th, Mr. F. E. Wentworth-Sheilds, M.Inst.C.E., presiding. A good discussion followed, which we publish below.—ED.

A CIRCULAR letter of inquiry on the subject of the Consistency of Concrete was addressed to the members of the Concrete Institute, in which it was suggested that a specification as drafted would be of service, pending experiments and tests that ought to be made to determine the exact proportion of water to be used in concrete in order to obtain the best mixture. This specification, as now slightly modified by the Committee, is as follows:—

> ***Consistency of Concrete.***—For mass concrete the quantity of water added to the other constituents shall be sufficient to make a plastic mixture which, after thorough ramming, will quiver like a jelly.
>
> For reinforced concrete the quantity of water added to the other constituents shall be such that the plastic mixture is capable of being rammed into all parts of the moulds and between the bars of the reinforcement.
>
> *Note.*—In dry or hot weather the quantity of water shall be increased in order to allow for evaporation.

Fifty-eight replies were received, from which a number of extracts are appended hereto.

The replies have been carefully considered by the Reinforced Concrete Practice Standing Committee, who have come to the following conclusions:—

1. It is inadvisable to lay down any definite rule as to the percentage of water to be used in mixing concrete, owing to the varying conditions which obtain. The proposed specification is difficult to improve upon, and seems to meet with general agreement.

2. The strength of concrete apart from any reinforcement increases as the amount of water used in mixing is decreased, this being more particularly the case during the earlier stages of the maturing of the concrete. Eventually the wetter of two mixtures will approach more nearly to the drier in strength.

3. In reinforced concrete, particularly in such portions as may contain a large amount of reinforcing bars or the like placed closely together, it is essential that the concrete should be sufficiently wet to pass between the reinforcing bars, and to thoroughly surround every portion of the steel. This should be ensured even at the expense of having the concrete wetter than would otherwise be desirable.

Where the reinforcement is not very closely spaced it is unnecessary for the concrete to be so wet.

4. Other conditions being the same, the drier the concrete the more quickly will it set and mature. This is of importance when there is any danger of green concrete being attacked by frost.

5. The wetter the concrete the greater is the tendency to contract during the process of setting and maturing. Appreciable contraction may sometimes continue for a period of several years.

6. The Committee is divided as to the advisability of determining by some means of mechanical test the exact degree of "wetness" or consistency of concrete after mixing. If some scale of consistency were adopted, it would be possible to specify that concrete for any particular portion of the work should be of such and such a consistency, after mixing. This would not, of course, be at all the same as specifying that any particular amount of water should be used in mixing such concrete, owing to differences of atmospheric temperature, aggregate, etc.

The advocates of the institution of some such scale of consistency are of opinion that the Concrete Institute should carry out tests on the subject.

SUMMARY OF REPLIES RECEIVED.

1. Several correspondents advocate the consideration of the results of tests before any rule is arrived at.

2. One correspondent suggests that a table should be given showing the maximum difference found in practice with different aggregates in the usual proportions and under different conditions, the quantity of water to be stated in gallons per cubic yard and the moulds assumed to be of soft wood. The form of table is as follows:—

PROPORTIONS OF CONCRETE.
Gallons of water per cubic yard of materials.

	Sandstones, Oolites, "Common" Brick.				Granites and Hard Limestones.	
	Aggregate, Dry.		Aggregate, Wet.			
	A.	B.	A.	B.	A.	B.
Dry weather						
Wet weather						

A = for getting into corners and sticking to steel all over to prevent corrosion.
B = for strength in masses of concrete.

3. Some correspondents point out that the quantity of water required might vary with the character of the cement—namely, whether "quick" or "slow" setting.

4. A correspondent points out that in one case 25 to 30 gallons of water per cubic yard of concrete has been advised, and in another case 21 to 24 gallons per cubic yard of dry material.

5. A second correspondent uses one gallon of water to one cubic foot of dry material where the aggregate is crushed Thames ballast; in his case, when the temperature has been above normal, it has been necessary to increase the amount up to 25 per cent. of the above-stated quantity, and when the reinforcement is heavy and ramming difficult, a further supply of water is necessary and 1½ gallons may be needed.

6. A third correspondent says that usually about 22 per cent. of the total volume of cement and sand or 20 per cent. by weight of these are usually taken for the quantity of water, but points out that about 15 per cent. by volume is required to enter into chemical combination with the cement and sand, and the rest is lost by evaporation, leaving in its place undesirable voids in the mass.

7. One correspondent suggests that the provision as to addition of water in hot and dry weather is unnecessary, for under such circumstances a certain increase would be automatically required to produce plasticity, and the loss should be prevented and not counteracted by means which tend to impair the quality of the concrete. He suggests the substitution of the following rule as sufficient to cover all cases:

"The quantity of water added to the cement and aggregate mixture shall be just sufficient to produce a plastic mass after thorough and complete mixing."

8. Another correspondent would prefer to substitute the following wording for the clauses put forward:

"For mass concrete as much water should be added as the mixture will take without spilling away or working up to the surface when the concrete is being conveyed to its destination.

"In the case of reinforced concrete if after ramming into position the water works up to the surface, the quantity may be considered excessive. Short of this, however, as much water as possible should be added."

9. A correspondent requires the condition "that when the concrete is thoroughly rammed into place water shall only just appear on the rammed surface."

10. A correspondent suggests the insertion of the word "light" before the word "ramming," as the heavy way in which this is carried out, especially in reinforced concrete, often results in the boards of which the moulds are made springing apart and so allowing the water and cement to ooze through the joints and detract from the final strength.

11. One correspondent suggests adding in the first paragraph the words "and not more than sufficient" after the word "sufficient."

12. It is suggested that emphasis should be laid on the fact that the mixture must only quiver like a jelly *after* the ramming has been completed and not before. It is also suggested that it might be advisable to state that where absorbent coarse materials are used great care should be taken to let them absorb all the water they require before being mixed with the cement, or having arrived by experiments at the amount of water which the aggregates will absorb that extra amount of water should be added at the time of gauging. It is thought, however, that the former practice would be preferable.

13. One correspondent points out that the words "quiver like a jelly" would apply to a small aggregate and gentle continuous ramming, but that a larger graded aggregate would not show the same result.

14. One correspondent does not favour ramming of concrete, preferring "a plastic mixture of the utmost possible density, which will flow into position in the moulds and round and in contact with the reinforcement (if any) without ramming other than consolidation aided by iron bars or spades."

15. Two correspondents point out that the danger to be guarded against where a plastic mixture is advised, is one of loss of homogeneity caused by repeated ramming resulting in the larger parts of the aggregate going to the bottom, leaving the fine particles at the top.

16. Another correspondent suggests that to the rule for reinforced concrete should be added the words "but in no case should the water be so much in excess as to cause the concrete to be of such consistency that when the mould is filled and rammed it has a distinct tendency to act as a semi-fluid under the punner."

17. A correspondent objects to the watering down of concrete to the consistency of slurry in order to make it run into the centering and round the steel, for the average centering is not sufficiently watertight to prevent a certain portion of the finest material escaping. He thinks that attenuated dimensions in reinforced concrete work should be avoided, so as to do away with the necessity for making the concrete so liquid.

18. It is suggested that the specification should state that for reinforced work the concrete should not contain so much water as to cause a large quantity thereof to exude during ramming.

19. It is pointed out that with reinforced concrete pipes it might be found impracticable to ram the mixture into all parts, and for such class of work it would have to be of such consistency as to run.

20. Several correspondents direct attention to the prevention of drying in hot climates. The procedure adopted by one correspondent is to use very little more water

in the original mixture, but to shade the work from the direct rays of the sun for the first twenty-four hours. Then if in small blocks they are totally immersed in a shallow tank of water, or if in mass concrete the work is covered with wet sacks or reed matting which is kept at the point of saturation. In either case the concrete is sprayed with water twice a day for about a fortnight.

21. Another correspondent in hot and dry weather waters the concrete two or three times daily for a week or so.

22. One correspondent suggests adding the words "and absorption" after the word "evaporation." He thinks that in hot weather it is possible the falsework should be watered on the outside unless a little extra water be added to the concrete.

23. A correspondent desires to call attention to the legal aspect of the case, which would probably be raised in the case of a dispute, and that is the "goodness" or "badness" of the rule. For this reason he thinks the personal element must be entirely eliminated, and the rule or specification should be so framed that the results will be the same irrespective of the persons who shall do the work. He suggests the making of a wooden box, 5 in. wide by 3 in. deep and 6 in. high, with two 1-in. square steel bars arranged vertically and attached securely to the 5-in. side, which latter is to be hinged at the bottom to the remainder of the box so as to be capable of being opened. In use the box would be filled, then after a specified length of time turned with the side carrying the bars uppermost and opened, when it would be found whether the concrete kept the correct form of the mould. He suggests that two boxes should be used, and that the Committee should consider—

(*a*) The size and shape of the boxes.
(*b*) The time before the first box is to be opened (minimum).
(*c*) The time before the first box is to be opened (maximum).
(*d*) The amount of ramming (preferably none).
(*e*) The degree of fulness of the boxes.

24. Finally, a correspondent calls attention to a different method of mixing concrete which he advocates. He first thoroughly mixes one part of cement with, say, two parts of very fine clean sand, using clean water enough to make a mixture of the consistency of thin cream, and then remixes with three parts of wet sand coarser than the first. This total mixture of 1 : 5 is then mixed with, say, five parts of broken stone. He asserts that with the proportions mentioned—namely, 1 : 10—the same strength is obtained as with that of 1 : 7½ concrete of the usual class.

The principle of the method is the obtaining firstly of thorough contact of the cement with every grain of the fine sand, which is rendered certain by the large proportion of the cement used, thus ensuring that every grain of sand is completely enveloped in the liquid mixture. A mixture of this consistency is well calculated in the remixing to adhere to all the wetted grains of the coarser sand, which will take up the superfluous cement and water and leave only that which sticks to the surface of the grains of the fine sand. He thinks that the fine sand mixture will thus more nearly fill the voids of the coarser sand than happens in the usual systems of mixing, where many grains of sand get washed by the water, while in other parts many grains of cement are found stuck together.

For a stronger concrete the mixture would possibly be 1 part of cement to 1½ parts fine sand, and this remixed with, say, 2½ parts of sand and 3 parts of broken stone. Another mixture would be 1 part of cement to 1 part of very fine sand; this remixed with 2 parts medium sand and again remixed with 3 parts very coarse sand; all finally remixed with 5 parts broken stone of different sizes.

This system only requires about the same amount of labour as the common one, because the liquid mixture is small in bulk and quickly made.

DISCUSSION OF THE ABOVE REPORT.

Mr. R. W. Vawdrey, A.M.Inst.C.E., Hon. Sec. of the Reinforced Concrete Practice Standing Committee.—With regard to this report he pointed out that the procedure adopted was for the Secretary of the Institute to issue circular letters to all the members and to some other persons whose opinions were thought to be of value, asking what their practice and advice were with regard to consistency—the amount of water which should be added to concrete in both massed concrete and reinforced concrete work.

The chief difficulty that the Committee experienced in discussing the question was that of describing the degree of wetness or dryness required. The amount of water to be mixed with the concrete could possibly be defined. That was, he thought, universally admitted; to mention a percentage—so much per cent.—of water as a general specification would obviously be absurd. It must necessarily vary with the class of materials used, with the quality of the cement, the temperature of the atmosphere during the actual process of mixing, the amount of ramming which had to be performed, and with the intricacy of the work. There were all sorts of conditions which might vary, but he thought it would be possible to devise some easy method of describing the degree of plasticity desirable in the concrete after mixing. That was suggested in one clause of the Committee's recommendations.

The Committee was very much divided on that question. A good many seemed to consider that it would be quite impossible to devise any such scale of consistency, and that even if that were done it would merely lead to difficulties on the part of the contractor, who would be expected to adhere to a particular scale on every part of his job—in fact, that he would have a lot more tests to pass. But that was not his view at all. The whole object of such a scale of consistency, to his mind, was that, if an engineer or an architect wanted to specify the degree of wetness or the consistency of the concrete in one part of a building, he might be able to do so easily. He might be able to say that in the beams, or any particular portion of any beam, he would like the concrete of such a consistency.

His own point of view, therefore, was that it would be very useful for the Institute to make some sort of tests which would possibly be adopted universally as a means of describing the consistency wanted.

Correspondent No. 10 remarked, referring to the ramming, that he wanted very light ramming only. Personally, he did not agree with that at all. Too much ramming could not be done, and the objection that heavy ramming often resulted in the boards springing apart and allowing the water and cement to ooze through the joints was not a fault of the ramming, but a fault of the shuttering. The shuttering should be so constructed that it could take proper ramming.

Correspondent No. 13 remarked that only concrete mixed with a small aggregate, and on which gentle continuous ramming had been used, would quiver, but that with a large aggregate you could not get the quivering. He entirely disagreed. In a large mass of concrete they certainly could get the quivering condition which everybody seemed to think desirable, even though the aggregate was very big. If they had a small reinforced concrete beam and a few big stones in it, they would not get any quivering, because the stones would wedge themselves from one side of the beam to the other; but in a heavy block of massed concrete they would get exactly the same appearance as in a small mass of very much finer concrete.

He disagreed entirely with Correspondent No. 14, who said he objected to ramming. He was convinced by theory and practice, and in every other way, that it was advantageous in every way to have thorough working and ramming of the concrete in reinforced concrete work, and in mass concrete work too.

Correspondent No. 24 raised a somewhat different point from that mentioned in the rest of the paper—namely, the proportioning of the concrete; but it was so very important that they considered it worth while putting it in this paper. He agreed entirely with his remarks, or with the idea which underlied them, which was that ordinary concrete was very badly proportioned. The strength of concrete obviously depended to a very great extent on the strength of the mortar. By mortar he meant the fine material in which the larger portions of stone were embedded and held together. An ideal concrete would be one in which there would be no mortar at all, or practically no mortar. He was, of course, exaggerating now intentionally. The extreme on one side would be a lot of particles of broken stone which could be so accurately fitted together that there were no voids between them, or practically no voids between them. That might be obtained if a large number of regular cubes of stone were accurately fitted together. The amount of mortar required in that case would be practically nil. On the other hand, the average regular stones—¾ in. stones—showed, approximately, 50 per cent. of voids.

The cost of the concrete could be very greatly reduced by the grading of the aggregate, so that the voids were very small—the total amount of voids was very small. That enabled the amount of mortar to be reduced; and it was quite possible to have a concrete consisting of, say, 4 of stone and—instead of the 2 of sand and 1 of cement which was originally specified—say 4 of stone, ½ of sand, and ¼ of cement. He had actually made concrete of those proportions which had been quite excellent; but, of course, that was dependent entirely on the fact that they could obtain an aggregate which had such a very small proportion of voids as those he had mentioned. In ordinary practical work one had to use materials that could be easily obtained, and in most cases it was certainly a fact that the stone which one got would have something like 50 per cent. of voids in it, in which case the proportions of 4 of stone, 2 of cement, and 1 of sand were about right. There was no objection to greatly decreasing the proportions of sand and cement if the amount of voids was decreased by proper grading and mixing of the aggregate. That, he thought, was the principle underlying the mixture suggested by Correspondent 24.

With regard to the general question dealt with in the report—the amount of water—he thought the general opinion of the majority of correspondents, and certainly of the Committee, was that the use of a large quantity of water was to some extent a necessary evil in reinforced concrete work. With massed concrete, in which voids were not so harmful at any rate as they were in reinforced concrete, it was undoubtedly possible to get considerably stronger concrete in its early stages by the use of concrete fairly dry; but in reinforced concrete, although assuming one could be sure that no voids would exist, even in that case too it would be advisable to use the concrete as dry as possible. Yet the risks of voids and improper contact of the steel with the concrete were so great that one was almost driven to use a concrete considerably wetter than would otherwise be desirable, and in many cases, such as the junctions of beams and columns, where there was necessarily a very large amount of reinforcement, it was practically impossible to use any sort of concrete otherwise than almost grout—slurry, as one of the correspondents called it. Although one might be forced, in spite of one's wish, to use concrete of that nature in certain proportions, at any rate there were many disadvantages besides the corresponding weakness of the concrete. One had been mentioned—that was the cracking. It did appear undoubtedly the case that the wetter the concrete was when it was moulded and placed in position, the more chances there were of contraction during the process of setting, and not only setting but during the process of maturing.

In his own experience, quite recently, he had a case in which contraction had obviously been going on in a floor of considerable area for three or four years, and was to some extent still going on. The amount was almost infinitesimal, but undoubtedly the movement of contraction was still taking place. He was not referring to a temporary contraction, but a permanent, maturing contraction, and he thought that was the experience of members of the Committee.

Mr. S. Bylander.—It would add very greatly to the value of this report if a definite quantity of water could be determined for, say, London practice or English practice for the standard concrete now accepted—1 cement, 2 sand, and 4 crushed gravel. He did not believe that the quantity of water would need to be varied so very much.

Mr. E. P. Wells.—With regard to the report on the consistency of concrete, it was a very well-known fact that the drier the concrete, and the more it was rammed, the greater the compressive strength; but it resolved itself purely into a question of labour.

With regard to reinforced work, especially if there were a number of small members within a beam, it was not advisable to do too much ramming. If the mixture were made too dry, they would never get the proper adhesion between it and the steel. Therefore it was advisable to make the concrete of such consistency that practically when it was in a heap it would not run. This meant that, if Thames ballast were used in the aggregate, and Thames sand for the mortar, about 7½ to 9 per cent., according to the time of the year, was the moisture required to be added; but if bricks were used, or sandstones, which would take up about 10 per cent. of water, then it was advisable to let a large aggregate take up all the water that it would contain before it was mixed with the sand and cement, which ought then to be mixed dry, after which about 6 to 7 per cent. of water was all that was necessary to give an absolutely perfect consistency to the concrete.

With regard to the bricks, it was an absolute necessity to wet the bricks first if they required to get a really good homogeneous concrete.

With reference to the use of the Fletton bricks in concrete, he called the attention of every member of the Concrete Institute to the great danger attending their use in the making of concrete. Two or three months ago, in some work that he designed for a gallery to a church, within a fortnight after the gallery had been finished it started to blow all over the place, and in every case where it was opened out it was found to be a piece of Fletton brick. Last week, at the meeting of the Committee, Mr. Davies showed some photographs of concrete which had been made of Fletton bricks, and within one week after it had blown all over the place and disrupted the concrete. Tests were made of the Fletton brick, and in that way was found an enormous excess of sulphuric anhydride. Altogether it was over 17 per cent. There was no doubt, now that Fletton bricks were becoming generally used in London, that they would have to be very careful if brick concrete were used that they did not get the Fletton bricks. He could not tell the age of the bricks, but Mr. Perkins, the District Surveyor, had seen a heap of Fletton bricks which had been in the open air for six months which had simply crumbled to pieces and been blown and disrupted in every quarter. A foreman of works had also said it was a known fact that Fletton bricks put into concrete would always blow.

With regard to concrete made in the slurry condition, it made an enormous difference in the strength. In one specimen which was made with nothing better than slurry he found it ran all over the place, and could not be collected at all.

If they wanted a cheap concrete, and to save labour, they must let the water do the work. If they were not particular as to labour, and could spend the money, then lessen the water and put in the labour. They would get a strength in seven days with a very dry mixture which would take a month with a moderately wet one; and if it was wanted to harden quickly, then the drier it was made and the more ramming it got the better, especially in wet weather.

Mr. D. B. Butler, A.M.Inst.C.E., said it must be borne in mind that in adding water to concrete the water had two functions to perform. The first was the chemical function of enabling the cement to set and to conform to certain conditions. The second was that the water acted as a flux which enabled the various constituents of concrete to flux into their places when they were rammed or shaken. The percentage of water required for a chemical combination was probably not more than 8 or 10 per cent., perhaps not so much as that—7 per cent. On the other hand, if insufficient water were used in gauging concrete to enable this flux to take place, the concrete would never flux in its proper place.

He congratulated the Committee on their common-sense recommendations, and agreed with them on No. 1, that it was inadvisable to lay down any definite rule as to the percentage of water that should be used. Some time ago he came across a case in which some concrete piles were made, and an attempt was made to gauge the concrete too dry in making these piles. The pile, when it was driven, fractured and split, and the contractor, of course, as usual, said the cement was bad. On examination of this pile and of the concrete, it was found that the concrete was so made that only about a quarter, or perhaps a third, was in contact with the reinforcement, thereby altogether nullifying the idea of reinforcement.

With regard to Mr. Wells's remarks about the great difficulty resulting from the use of Fletton bricks, he really did not quite understand how there could be 17 per cent. of sulphuric anhydride in the bricks; 17 per cent. of sulphuric anhydride would correspond to somewhere about 25 per cent. or more—30 per cent., perhaps—of calcium sulphate.

Mr. Wells: 22½ per cent. of calcium sulphate.

Mr. Butler: He could not conceive any brick standing for even a month exposed to wet with all that amount of calcium sulphate in it.

They all knew that calcium sulphate, or plaster of Paris, was absolutely non-hydraulic, and went to pieces in water. It was a well-known fact, however, that if there were much calcium sulphate in the aggregate or in the cement it would cause blowing, and he had more than once come across cases in which old plastering had been used in making the aggregate in walls, and it had caused disintegration of the concrete.

Mr. P. H. Fraser, A.R.I.B.A.—With regard to the consistency of concrete, a subject in which he was very much interested, the last speaker had mentioned piles which were mixed too dry. He had a case, two or three weeks ago, of five piles which were wanted in a great hurry. They were mixed very dry. When he approved of the consistency of the concrete the contractor was quite aghast. He said: "It looks like the inside of a Stilton cheese, and I am sure it will fall to pieces." These piles were driven in twenty-six days. After making, they were driven with a 30-cwt. monkey with a 5-ft. drop, and, although the heads fractured rather, there was not the slightest appearance of fracture at any part of the pile except just under the head.

The word "ramming" presented to his mind the action of striking heavy blows with a heavy tool, and in reinforced concrete he thought this was quite wrong. They could not get that artificial strength which they all knew concrete got which had consolidated during its setting. They could not get that by ramming; they got disruption more than increased strength. The only ramming he should recognise consisted of poking with an iron rod, and nothing in the shape of a heavy weight.

He was very strongly in favour of a dry concrete. The Committee, although they did not actually uphold wet concrete, seemed to imply that the wetter a concrete was, within certain limits, the better. He took the view that the drier it was, within certain limits, the better. The moral of paragraphs 2, 4, and 5, with regard to the ordinary strengths of wet and dry concretes compared, was to use your concrete dry and get your ultimate strength as soon as you could, putting on one side the fact that wet concrete would never get the same eventual strength as a dry concrete; and where the reinforcement was particularly complicated, they did not get over that complication by using a wet concrete, whereby a sort of mud got underneath the bars and a kind of mass of stones got on top of the rods. They wanted to use a fine concrete at these points to bring it down to quarter-inch stuff, or half-inch at the outside, and supervise at those points as much as they could supervise and see that they were properly plumbed. One of the correspondents inquired whether a greater amount of water was required for a quick or slow setting of cement. He had not had any experience of that, and he should be very glad if any gentlemen there would give them any information on that point.

A suggestion was put forward that sand and cement should be mixed dry and then wet, and then mixed with the aggregate later on. Anybody who had seen that process actually carried out would bar it for all time. It was a most painful thing to watch; it cost 5s. a yard more for labour, and he was sure the result, except in perhaps one or two very elaborate mixing machines, must be very bad.

There were a few rough-and-ready methods of telling whether the consistency of concrete was right. If they loaded concrete into a barrow, by the time it was wheeled into its place it should not have taken a horizontal surface. They should be able to take a shovelful of concrete from the bank and hold it at a slight angle, and it should show no signs of the cement dropping away. Also they should be able to make a hole in the concrete in the barrow which no amount of ordinary vibration should ever quite cause to fill in.

Mr. Thomas Potter said where the aggregate was of one kind and consistency there was obviously no difficulty in ascertaining the amount of water required, assuming that the aggregate was not some dry and some wet. In the ordinary way there were many difficulties to contend with. The aggregate might be of a porous nature and absorb a good deal of water, or it might be gravel which would practically absorb none. Then, a small aggregate required more water than a larger one. It might have to be washed, and would then require a smaller amount of water than if dried. It depended, too, upon the purpose the concrete was for.

For monolithic walls it could not be rammed or compressed between the forms to much extent, or the form boards might be pushed out of place. For floors it could be beaten or compressed. As a result, more water was required for concrete for walls than for floors.

For floor purposes, if too much water were used, it came to the surface rapidly when beaten or compressed, and cement adhered to the surface of the beater. Violent beating of the concrete, except when it was in large mass form, caused the water and cement to quickly come to the surface, which was avoided if the impact was of a general character and quickly performed.

The amount of water necessary was very difficult to standardise for general practice. Generally he would prefer the personal equation in determining the amount, assuming a capable man directed the work.

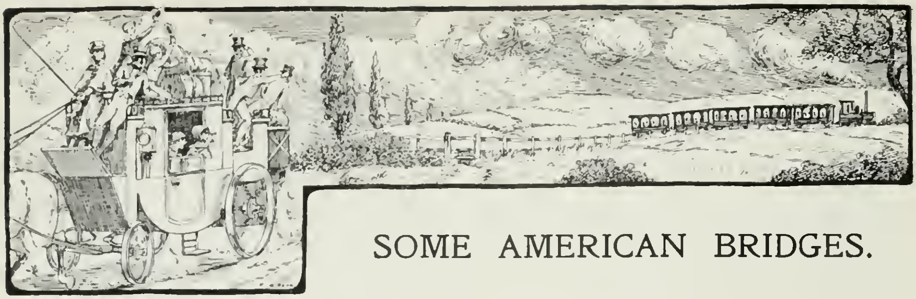
When the aggregate had to be washed previous to, and at the time of, use, and was of a non-absorbent character, but containing mud or clay, as gravel from sluggish streams or pits where it contains clay, a small amount of water was necessary for mixing purposes.

Mr. E. Fiander Etchells, F.Phys.Soc.— On page 2 of the report it was said that the strength of concrete, apart from any reinforcement, increased as the amount of water used in mixing decreased. The Committee were quite right in not building anything on that definition, because it was so obviously inaccurate.

He had tried to co-ordinate the views of all the correspondents and of the Committee. There seemed generally in the minds of each of them a collective idea as to what the consistency should be, and the difference appeared largely to be in the matter of words. Therefore, he suggested the following for consideration: "For reinforced concrete the amount of water to be added should be sufficient, but not more than sufficient, to produce a plastic and quivering mass which will tremble and exude a small amount of water after tamping."

The Chairman (Mr. F. E. Wentworth-Sheilds).—The second sentence in the specification which apparently had received the Committee's blessing did not seem to him to convey anything at all. It said that the quantity of water should be such that the plastic mixture was capable of being rammed into all parts of the moulds and between the bars of the reinforcement. That rather reminded him of an old foreman to whom he propounded the question, what was the correct quantity of water to use in concrete, and he replied, in a very serious and thoughtful manner, "Well, sir, the great thing is not to put too little and not to put too much," and he seemed to think that had solved the matter. Perhaps they could have a definition which would settle the question a little more closely.

Mr. R. W. Vawdrey, A.M.I.C.E., replying to the discussion, thought every member of the Committee would be inclined to agree with the Chairman's criticism. The attempt to answer the question as to how wet the concrete should be had, of course, failed. That they must admit. His own view was that some sort of scale of consistency should be evolved, if possible, by the Institute which would make it possible to refer to a particular degree of consistency of the concrete. The general feeling was that wetness of concrete was an evil, but in some cases it might be a necessary one.



SOME AMERICAN BRIDGES.

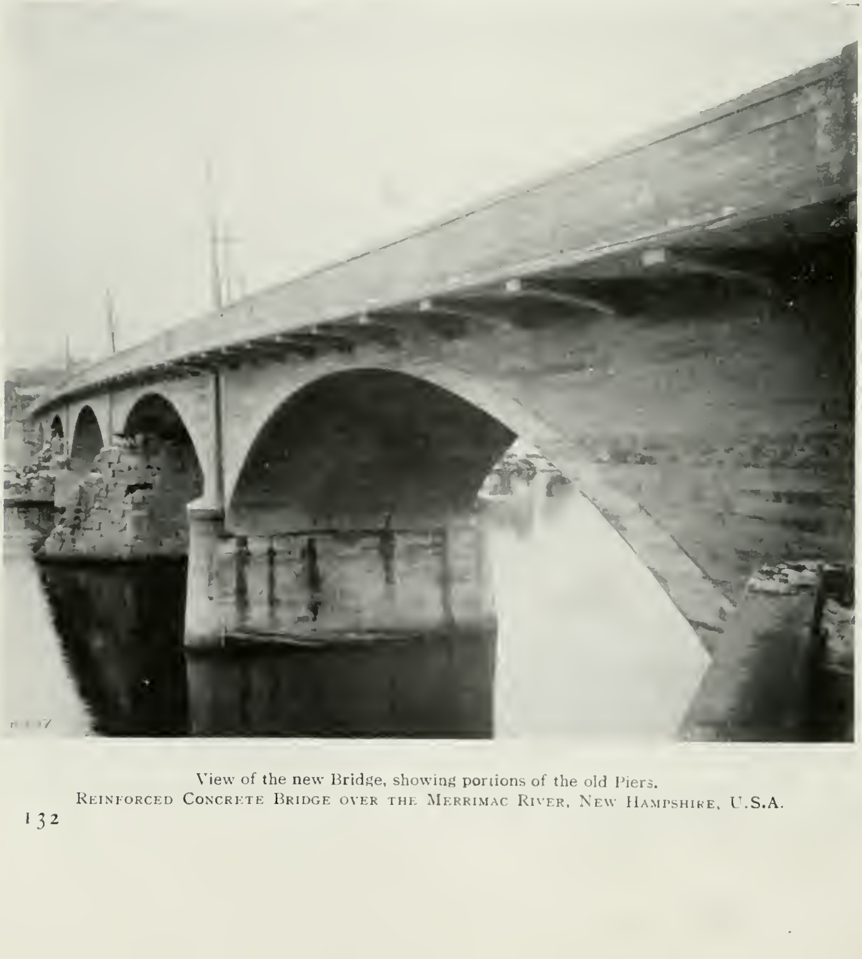
The following examples of reinforced concrete bridges in the United States are striking testimony to the rapid development of that material for bridge construction.—ED.

REINFORCED CONCRETE BRIDGE OVER THE MERRIMAC RIVER, NEW HAMPSHIRE, U.S.A.

Bridge built under difficult conditions.

THE reinforced concrete bridge over the Merrimac River described in this article was built to replace an old iron bridge which, while in fairly good condition as to the metal in the trusses, did not possess the requisite strength in the floor and details for modern loadings.

The old iron structure having been condemned as unsafe, a committee was formed with authority to build a new bridge. Plans and specifications were at first proposed



View of the new Bridge, showing portions of the old Piers.
REINFORCED CONCRETE BRIDGE OVER THE MERRIMAC RIVER, NEW HAMPSHIRE, U.S.A.

for a new steel bridge, making use of one of the old piers and abutments. After much discussion this plan was abandoned, as the voters of the town took a strong stand against a new steel bridge, arguing that, as the old structure had only lasted twenty-eight years, it was poor economy to build a new one of the same material. It was finally decided to erect a reinforced concrete bridge, and Mr. W. M. Denman, Consulting Engineer, of Springfield, Mass., U.S.A., was commissioned to prepare plans and specifications, and these were finally accepted. The following particulars of the work are taken from an article in *Engineering Record* by Mr. Denman.

When the replacing of the old bridge with a new steel structure was contemplated it had been intended to build a temporary bridge on piles, but by using concrete arches it was possible to maintain traffic on the old bridge while building the new one underneath. This was rather difficult as the grade of the roadway of the new bridge was required to be elevated 4 ft. above the road level of the old iron structure.

The old iron spans were 150, 168 and 170 ft. long, with an 18-ft. roadway and one footpath. The portal struts were very light, being only 18-in. deep, made of the lightest

View of Finished Bridge.

Reinforced Concrete Bridge over the Merrimac River, New Hampshire, U.S.A.

angles and lacing bars. In order to build the new bridge under it, it was necessary to raise the old spans, and the light portal struts made this a very difficult undertaking. The middle span was raised 7 ft. above its original grade, and the shore ends were put up 5 ft. The struts, however, began to buckle, and the bearings of the end-posts displayed a tendency to work out of line before the spans were half-way up. The light struts were stiffened by embedding them in concrete. Forms were built about them and concrete poured, while the traffic passed continuously over the structure causing heavy vibration of all members. Nevertheless the concrete set perfectly.

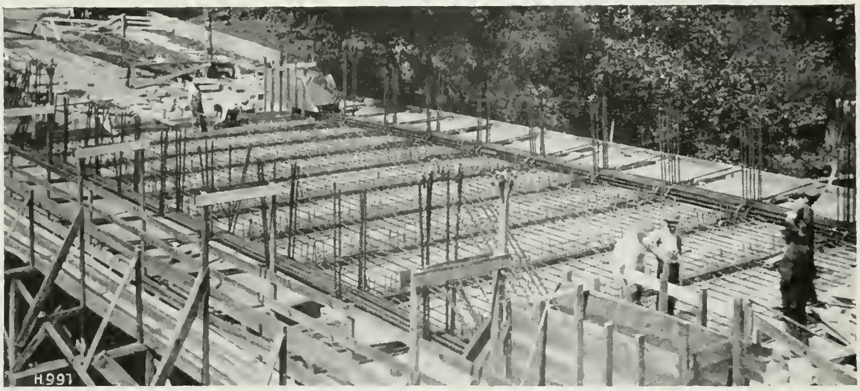
One abutment for the new bridge rested on rock just within the old abutment, which is used as backing to resist the thrust. It was found necessary to underpin this abutment in sections with concrete.

Two of the piers also rest on rock at 4 ft. to 6 ft. below low water. The two other piers are on piles in 10 ft. to 15 ft. of water.

The piers of the new bridge were so spaced as to permit of their construction without interfering with the old piers. The arches of the new spans necessarily passed through the spaces between the tops of the old piers and the old truss bridge. This was done by boxing in the 4-ft. by 4-ft. columns of blocking under each end-post and concreting the arch ring around the box, diverting the steel reinforcement at that point. After the arches had hardened, the fill was made and the old bridge let down on the new, so that it could be dismantled and removed. The columns of blocking were taken out and the holes plugged with concrete.

The new bridge has spans of 88 ft., 94 ft. and 100 ft. The roadway is 30 ft. wide in the clear, and the footpath is 6 ft. wide.

The exterior surface of the concrete was finished by rubbing the arch ring, coping and balustrade, the intervening spaces being either brushed or dressed by pneumatic tool.

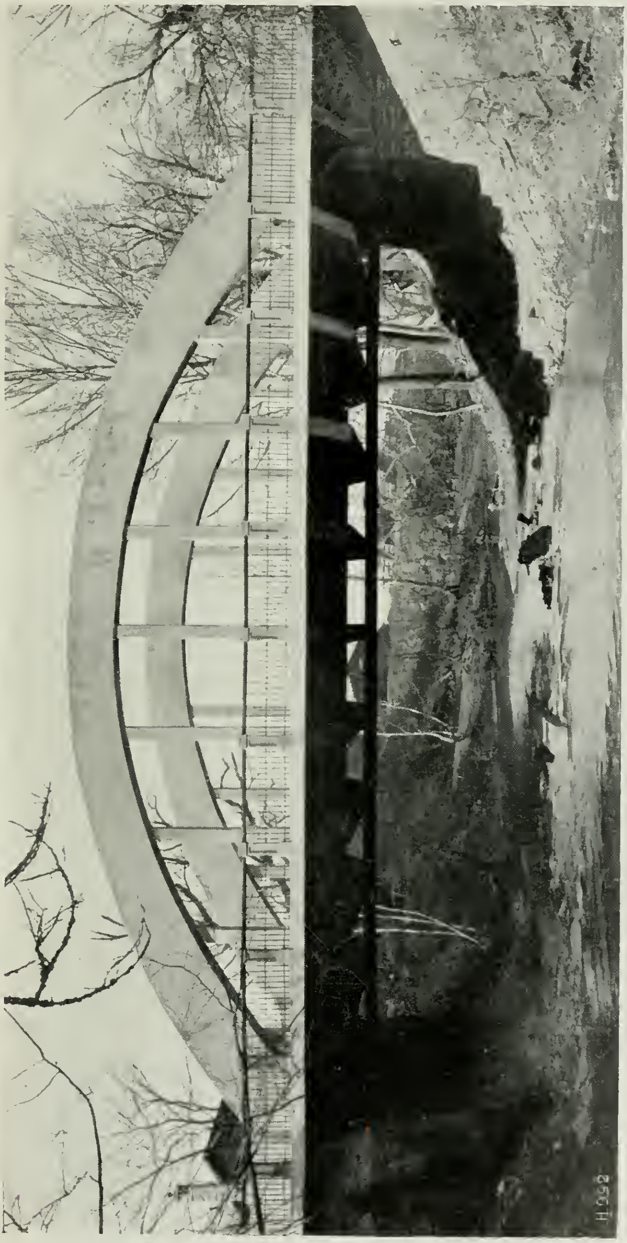


View showing Reinforcement for Floor of Bridge.
Reinforced Concrete Arch Bridge near Cincinnati, U.S.A.

REINFORCED CONCRETE ARCH BRIDGE NEAR CINCINNATI, U.S.A.

This bridge was erected by the Hamilton County Commissioners, whose policy it is to rebuild bridges in the county of reinforced concrete as fast as the old ones of wood and iron become unfit for use. It is situated about eight miles north of Cincinnati, and carries Benson Street, the main thoroughfare between the villages of Reading and Lockland, Ohio.

The construction of the new bridge was described by Mr. E. A. East, Deputy County Surveyor, Hamilton County, Cincinnati, Ohio, in our contemporary *Engineering News*, from which article the following particulars are taken.



View of Finished Bridge.
Reinforced Concrete Arch Bridge near Cincinnati, U.S.A.

In looking over the site of the bridge it was found that the present waterway opening must be preserved, on account of the large amount of water coming through the creek at flood periods. This fact prevented the use of any of the usual types of arch bridges.

The bridge consists of two hingeless reinforced concrete arch ribs, built entirely above the roadway, the thrust being cared for by tension steel in the plans of the floor tied thoroughly around the steel in the ribs. The roadway is suspended from the ribs by nine hangers of reinforced concrete, the steel rods, hooked around rib reinforcements above and floor rods below, taking all the stress. This particular type of arch was developed because in the preliminary studies it was found that the old masonry abutments were in such good condition and of such size as to make them amply able to take the load of the new bridge, provided the weight could be distributed uniformly over their width. The tied arch causes only vertical load on the abutments, and to distribute this vertical load the heavy transverse beams below the arch ends were designed.

The arch ribs are 3 ft. wide at the springing line and 3 ft. wide at the crown. They are 4 ft. 10 in. deep at the springing line and 3 ft. 6 in. at the crown, with a uniformly decreasing section from the spring to crown. The floor is of slab and transverse girder design with cantilever sidewalks, each girder being attached to two transversely opposite hangers. These cantilevers give a total width of 50 ft. to the bridge, although the abutments are only 36 ft. wide.

No provision was made for a movement at the abutments due to temperature changes, and the transverse beams at the abutment were laid directly upon the old masonry. The arch ribs, however, were designed to care for all temperature stresses. Although the bridge has now experienced some of the hottest as well as coldest weather of this climate not a single crack has developed.

The wind bracing, and also the long column effect in the arches, is taken care of by the hangers, the steel in these hangers being placed near the short sides to reinforce them for this purpose. It was found that there is a very large horizontal shear in the arch rib at the haunches equal to the total horizontal thrust. This shear lies in a horizontal plane just above the tension reinforcement, and must be thoroughly taken care of. This shear does not appear in arches where the thrust is taken by solid masonry foundations.

The bridge is designed to carry two 40-ton street cars. The maximum stresses allowed were 16,000 lbs. per sq. in. for steel and 600 lbs. per sq. in. for concrete in compression.

A novel plan was decided upon in erection. The floor and haunches of the arches were built on a heavy centering, then the hangers were erected and cast in concrete, and finally the arches were built in place. On account of this method of construction it was necessary to build very heavy centering.

The concrete for the floor was poured in sections. Each girder and its slab extending halfway to the next girder on each side was cast monolithic in order to obtain the T beam effect.

After the entire floor was finished the steel for the hangers was erected, forms built around it, and the concrete poured. This gave a rigid support for the building of the arch forms.

The reinforcing was entirely of cold twisted steel, the bending all being done on the site. Considerable trouble was experienced in trying to place the steel in the girders owing to their depth; this, however, was overcome by assembling the steel frame and wiring it thoroughly together above the girder form, and then lowering it into place with block and tackle. The remainder of the reinforcing offered very little resistance to rapid placing.

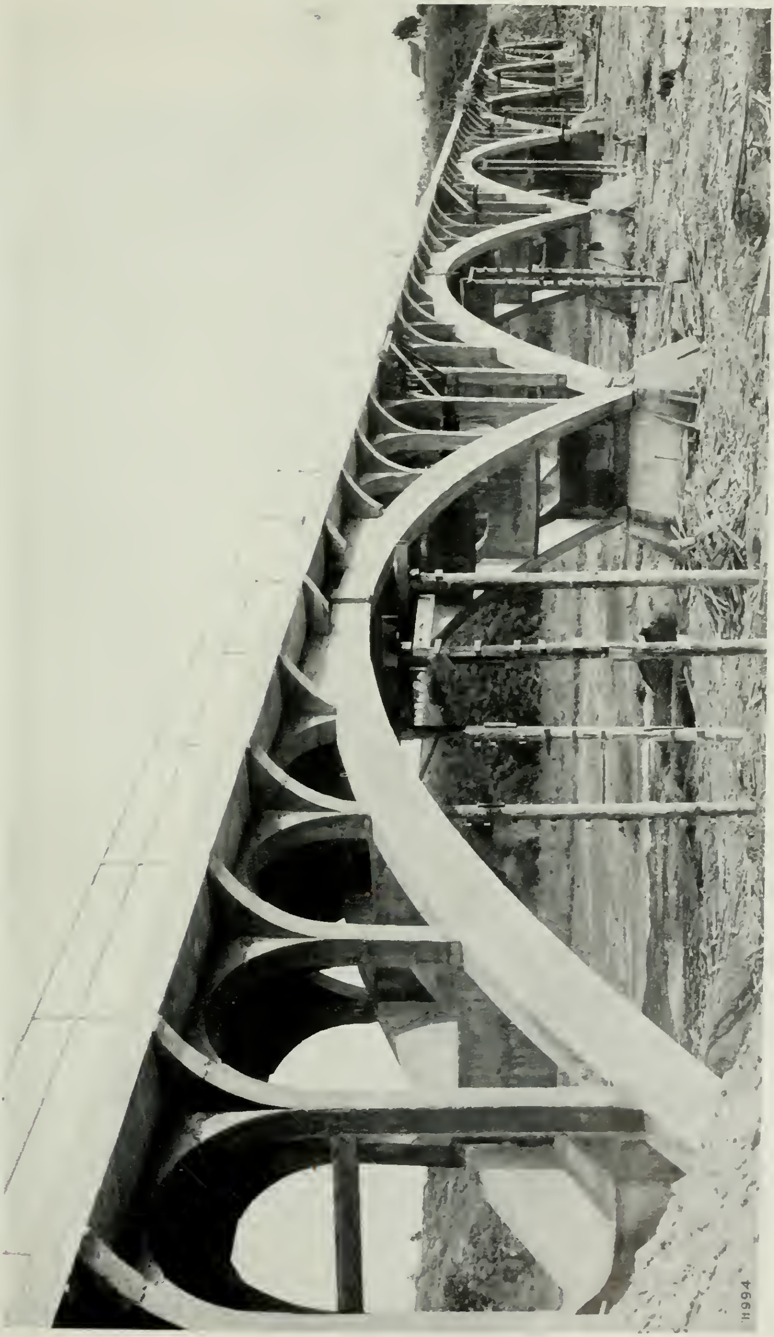
The lampposts are of concrete cast in place, the wiring being placed in loricated iron tubing concealed in the concrete.

REINFORCED CONCRETE BRIDGE IN CALIFORNIA, BUILT IN SECTIONS ON GROUND AND ELEVATED TO POSITION.

THE Young Construction Company of Los Angeles has recently completed a 700-ft. reinforced concrete bridge of three huge arches over the San Luis Rey River at Oceanside, California.

A unique feature of the bridge here illustrated lies in the fact that all its several members were built on the ground, where they were left until the concrete was sufficiently set and then hoisted into position. The arch ribs were poured in two sections and the spandrel posts cast separately. After these were put in place on piers and abutments the forms were bolted in place for the deck. By this method the excessive cost of falsework was saved.

The San Luis Rey bridge is the second of its kind erected in the State, the first one having been built over three years ago at Santa Cruz by the same engineers.



Reinforced Concrete Bridge in California Built in Sections on Ground and Elevated to Position.

The San Diego County bridge, however, incorporates later developments and improvements in design and method of construction. It was designed on the Thomas system, on which patents have been secured by Mr. W. M. Thomas, C.E., of Los Angeles.

The bridge was designed to carry a load of 250 lbs. to the sq. ft. It consists of six spans, four of which are 103 ft. in length, the two end spans being 107 ft. The bridge is 19·4 ft. in width over all, with 18 ft. roadway. The foundation for piers and abutments consists of thirty piles to each pier and abutment, the piles being cut off 2 ft. below mean low tide, the foundations thus extending down about 3½ ft. below the river bed. Each pile was calculated to sustain a 17-ton load. The foundations of piers are 12 ft. in width and 26 ft. in length. The concrete used in piers and abutments was 1 : 3 : 6. There is no reinforcement in piers. In the arch ribs the concrete aggregate was 1 : 2 : 5, beach gravel also being used. The arch ribs were made on the ground under the respective positions which they were later to occupy in the completed structure. These arch ribs or beams are 11 ft. on centres, and are reinforced with a steel framework consisting of twelve ¾-in. round rods and ¼ × 1¼-in. flats, the reinforcing being connected at the crown end of the beam to cylindrical plates of steel having a ball joint mated into a cup in the opposite beam, the semi-spherical protuberances. At the reverse end of the beam are ½-in. semi-circular plates forming portions of the hinges at skew-back on the piers. The hinge shoe on piers consists of 1-in. bolts, the rods extending deep into the concrete.

The spandrel posts, also made separately on the ground, were poured in 12 × 16-in. sections, the concrete aggregate being the same as in the arch ribs, that is 1 : 2 : 5. The longer posts are reinforced with four ⅝-in. round rods; the shorter posts have four ½-in. rods. These rods were left protruding from the sides, top, and bottom of the posts. The rods extending from the top form a tie to connect the spandrel arches with the spandrel posts; the rods protruding from the bottom have a thread, and they extend through the rack beam and bolt the spandrel post to the beam; the rods extending through the sides of the beam are embedded in diaphragm cross braces and hold the posts to a rigid position.

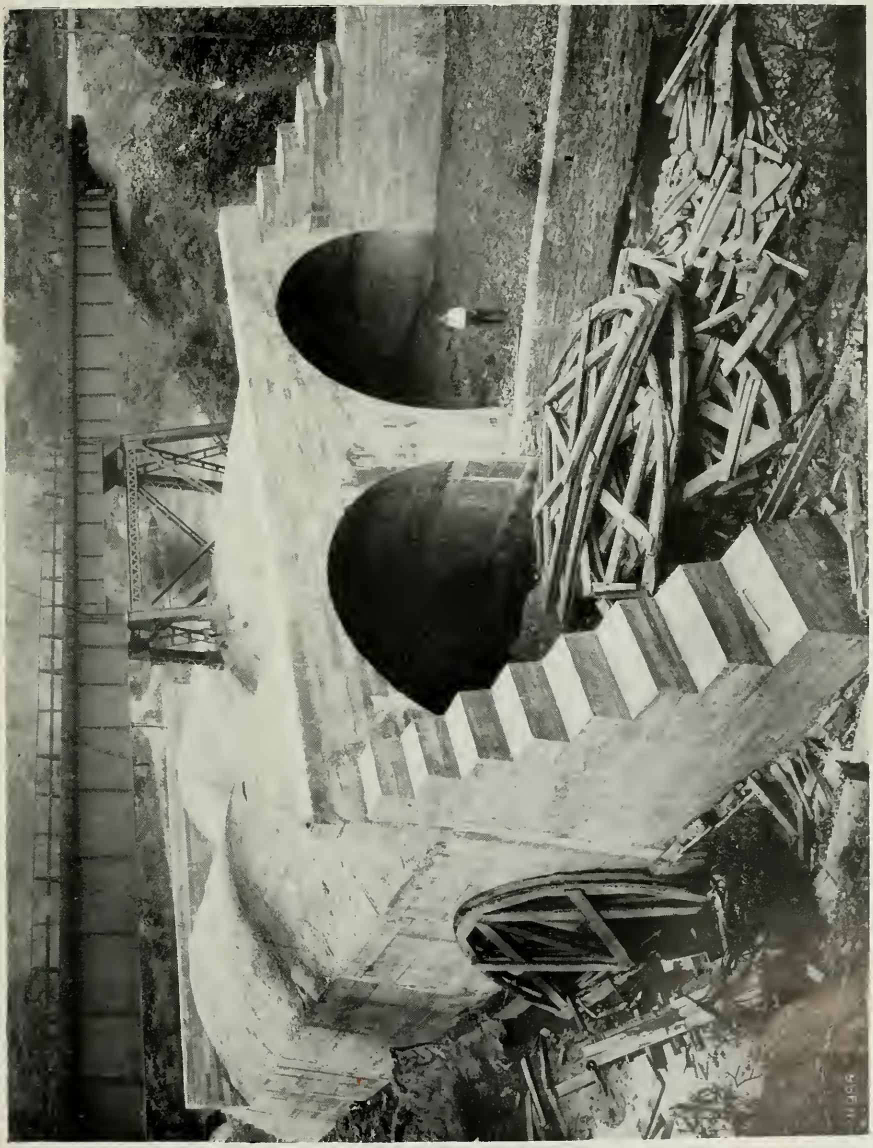
Holes are left in the spandrel posts and the wooden centering for spandrel arches to provide means for firmly bolting the floor supports to the spandrel posts; the monolithic concrete deck was then poured in the usual manner. The floor was reinforced with ⅝-in. round rods running transversely across the bridge, and the outside cantilever portion of the floor has rods running longitudinally 6 in. on centres.

CONCRETE ARCH BRIDGE FOR THE BESSEMER AND LAKE ERIE RAILROAD CO., U.S.A.

THE increase in the weight of motive power has augmented the standard loading on the Bessemer and Lake Erie Railroad, and a second track has been constructed, or is under construction, over the entire road, and wherever practicable concrete arches are being used to replace steel viaducts. The construction of this second track was the reason for building the concrete arch spans at Rural Ridge Station described below. The concrete masonry work was designed and built under the direction of Mr. H. T. Porter, chief engineer of the Bessemer and Lake Erie Railroad Company, and the T. A. Gillespie Company of New York and Pittsburg were the contractors.

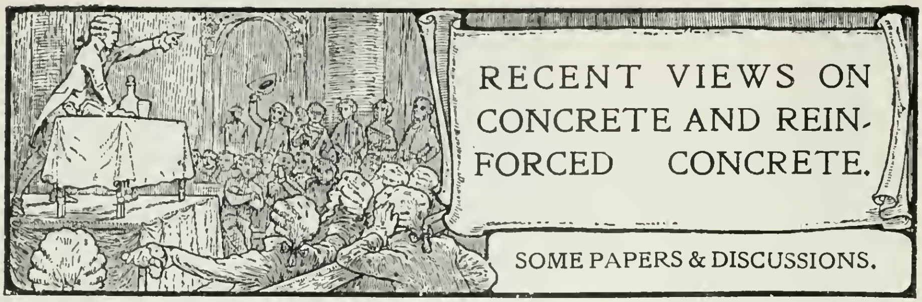
The original steel structure was 240 ft. long, consisting of three 60-ft. girder spans carried on the abutments and two intermediate steel towers.

The arches are semi-circular, of 26 ft. span, carried on side walls rising 5 ft. above the foundations. Upstream are three straight wing walls 24½ ft. long, the straight walls being adopted to facilitate future increases in length. There is no reinforcement, and the arch is 2 ft. 3 in. thick at the crown. The foundations, 8 ft. wide for the centre pier and 8 ft. 7 in. for the abutments, were carried down to rock, giving a depth of 10 ft. The total amount of concrete was 3,284 cu. yd.



Concrete Arches for the Bessemer and Lake Erie Railroad, U.S.A.
Work in progress.

Before work was started a temporary highway was opened north of the site, and a temporary creek channel was dug where necessary to the south. The arches were then built in three sections and completed, with the exception of the south wing wall



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

SOME PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

INSTITUTION OF CIVIL ENGINEERS.

REINFORCED CONCRETE WHARVES & WAREHOUSES AT LOWER POOTUNG, SHANGHAI.

Paper by MR. S. H. ELLIS, M.Inst.C.E.

The author described works recently constructed, under his supervision, beside the Whang-Poo River, near Shanghai, in North China. These comprised a reinforced concrete piled wharf, 1,160 ft. long by 174 ft. wide, with a minimum depth of 21 ft. of water at its face; a reinforced concrete quay wall, 495 ft. long and 21 ft. high; and two reinforced concrete four-story warehouses, each 300 ft. by 100 ft. in floor area; as well as offices and staff-quarters, sheds for temporary storage of goods, and a power and light installation.

The question of foundations was dealt with at some length in the paper, the site consisting of river-deposited silt to an unexplored depth. The wharves and lighter buildings were founded on groups of reinforced concrete piles. The warehouses rested on a raft of reinforced concrete (girders, beams, and foundation-slab), connecting the column-bases with a grill 4 ft. 6 in. to 5 ft. deep.

Wharf.—The piers of the wharf are 15 in. square and spaced 15 ft. from centre to centre in each direction; each is supported on a group of four 14-in. piles. Girders run across the wharf from side to side over each row of piles and support a system of cross-beams spaced 5 ft. apart between centres, and a continuous reinforced concrete deck 5 in. thick. The pier-bases are connected both longitudinally and transversely by horizontal members, the wharf being further stiffened by a system of diagonal bracing from front to back at each row of piers, and also longitudinally for the last two bays at each end.

Over the greater part of its length the wharf is covered in up to 52 ft. from the face by sheds of steel and corrugated iron, the piers which carry the stanchions for these being borne by a large number of piles. In line with the front of the shed a wall of reinforced concrete sheet piles is provided throughout the length of the wharf, and is continued upwards to above high-water level by a curtain of reinforced concrete slabs; this acts as a retaining-wall to the silt.

The piles, of which there were 4,000, were moulded in a horizontal position. With the exception of the pile-caps, beams, and deck, all the wharf members were moulded on shore, thereby avoiding for the most part the operation of concreting below high-water level.

The most favourable anticipations have so far been realised by the behaviour of the wharf in practice, when it has been subjected to the test of having considerably more than the estimated maximum load stacked upon it locally, without any appreciable settlement in the superstructure.

Quay Wall.—The quay wall, 495 ft. in length, was constructed of concrete throughout. It was founded on cross rows of three 12-in. square piles, of which the front row served as king-piles to a continuous line of sheet piles which formed the face of the wall to a height of 12 ft. below quay-level. The heads of the piles were connected by a deck 14 ft. wide, above which the wall consisted of a nearly vertical slab tied back to the deck by a series of triangular counterforts.

Warehouses and Office Buildings.—Each floor of the warehouses was designed to carry a superficial load of 500 lb. per sq. ft. throughout its area. The entire structure was of reinforced concrete, except the panel walls, which were of brickwork 15 in. thick.

The foundations of the office buildings were carried on piling, a maximum load of 11 tons per pile, as in the wharf, being allowed.

Concrete.—The concrete throughout was composed of 1 part Portland cement, 2 parts sand, and 4 parts beach-gravel, except for the foundations of the buildings, where the proportions were 1 : $2\frac{1}{2}$: 5. In all cases the cement was tested on the site for tensile strength, etc., according to the British Standard Specification.

Reinforcement.—The greater part of the steel in the reinforcement was in the form of plain round bars and wire ties. Expanded metal of $\frac{1}{4}$ in. by $\frac{3}{16}$ in. section, and with a mesh 3 in. across, was used for all floors of buildings and for the deck of the wharf. The steel was specified to be capable of bearing a tensile stress of between 60,000 lb. and 70,000 lb. per sq. in.

The interior first-floor columns were designed to carry a *maximum* load of 376 tons. They were octagonal in section, and the $2\frac{1}{4}$-in. vertical bars were kept in place by a series of inner circular bands to which the vertical rods were fastened by means of U bolts. The outer hooping was a continuous rod $\frac{1}{2}$ in. in diameter, wound spirally with a pitch of $2\frac{3}{4}$ in. The second and third floor interior columns were of similar design, but of lighter section and reinforcement. The roof columns were of ordinary rectangular section, with wire ties 6 in. apart.

All exterior columns above the first floor were reinforced by the latter method, continuous spiral winding, with its accompanying high unit stress on the concrete confined within it, being necessary only below that level. The interior columns of the first three floors were protected from accidental injury by being encased in $\frac{1}{8}$-in. steel plates up to a height of 6 ft. The roof columns were fitted with angle bars at the corners for the same purpose. All steel was covered with $1\frac{1}{2}$ in. of concrete, except in the case of floors, where this dimension was reduced to 1 in. In structural members exposed to the weather a cover of 2 in. was given in almost all instances. The absence of rust marks on the concrete surface of the wharf, and the fact that wherever bared the steel was clean, seemed to show that this coating of good dense concrete was an effective preventive of corrosion, even when the structure was alternately wet and dry.

Special care was taken, both in designing the systems of reinforcement and in preparing the steel skeletons, to ensure that all steelwork was rigidly fastened together and in its proper position with reference to the moulds before concreting was started, and that no loose parts were omitted by accident or knocked out of place during the process of ramming the concrete. With this end in view the wharf girders, which ran across the structure every 15 ft., were reinforced with what was in effect a pair of light skeleton girders, built up of $2\frac{1}{2}$-in. angle bars at the top and of $2\frac{1}{2}$-in. by $\frac{5}{8}$-in. flat bars at the bottom, shear members being riveted to them.

A corresponding system of rigidly fixed shear members was obtained in the wharf deck beams by the adoption of double Kahn bars, 4·8 lb. per lin. ft., in conjunction with a single round bar at the top and bottom. In the more heavily loaded beams and girders of the warehouses nothing but plain round bars and wire ties were used. The greater part of the shearing stress was taken up by the tension bars, which were bent up at intervals to pass along the top of the member over the points of support. In order to resist the remaining shear, to confine the concrete, and also to link together the main reinforcement, three rows of stirrups were employed. These were made of two wires, either $\frac{1}{4}$ in. or $\frac{5}{16}$ in. in diameter, twisted together, with their free ends clenched tightly round the longitudinal bars at top and bottom. The sets of bars were tied together horizontally with wire links, and the whole skeleton was tightened and stiffened by struts $\frac{1}{2}$ in. in diameter at intervals of 3 ft. to 5 ft., which converted the bars and ties into a rigid framework.

The author was engineer, in company with Messrs. Davies & Thomas, architects, of Shanghai. The contractors were Messrs. Vering, of Hamburg.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

FOUNDATIONS FOR HOVIS MILLS, VAUXHALL BRIDGE ROAD, LONDON.

THE ground at the site of the new Hovis Mills, Vauxhall Bridge Road, London, being rather unreliable, the question arose of a suitable foundation, carried down to the deeper firm strata of hard ballast.

Three methods of constructing the foundations were considered—viz., a reinforced concrete raft, timber piling, and C.P. (cast in place) concrete piles.

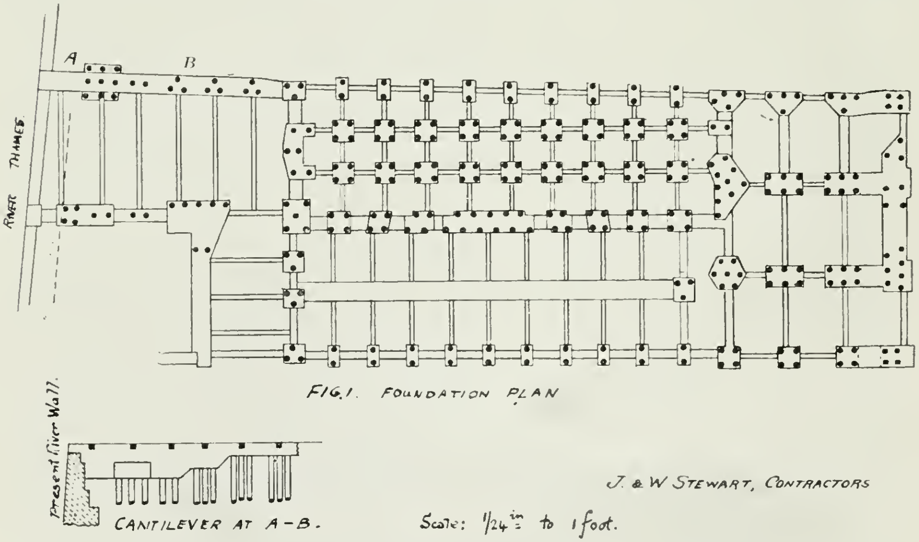
The last method proved in this case the most economical and the speediest, and had also the advantage that the rather uncertain lengths of the piles required did not affect it, as under the system of C.P. concrete piles used each pile is constructed just of the length required in each case.

At the part of the building facing the river the piling could not be brought up close to the front on account of the present quay wall, and the front end of this building had therefore to be carried on heavy reinforced cantilevers, as shown in the drawing on this page.

At the opposite end of the buildings the warehouse is built close against some rather old buildings, and some difficulty was anticipated in piling so close up to the existing walls, but the work was carried through without the slightest trouble, although some of the piles were within only 9 in. of the existing walls.

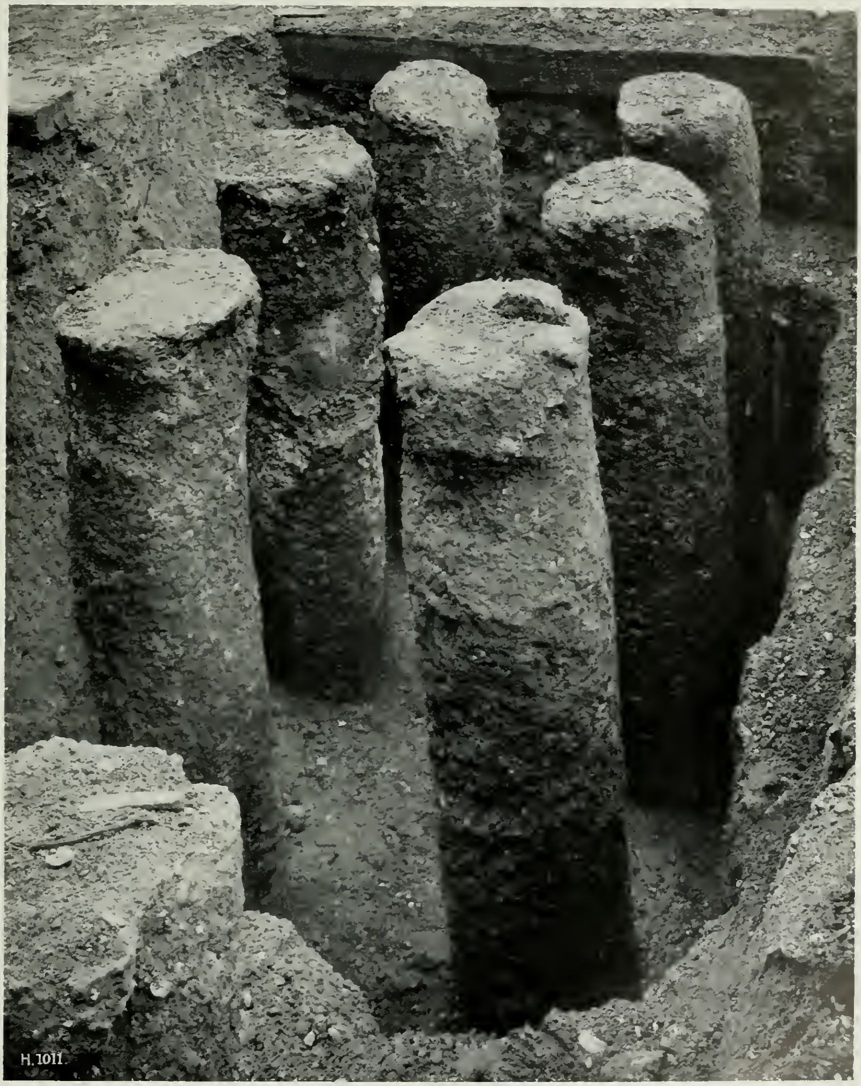
The piles varied in length from 17 ft. to 33 ft., and were all reinforced with a steel joist or with a cage of six ⅞-in. diameter steel bars, tied together at short intervals with steel hoops.

The architect for the work was John Clarke, Esq., F.R.I.B.A., of 34 Castle Street, Liverpool, and the foundations were designed and constructed by Messrs.



Foundation Plan.
FOUNDATIONS FOR HOVIS MILLS.

J. and W. Stewart, foundation contractors, of Caxton House, Westminster, on their system of C.P. concrete piles.

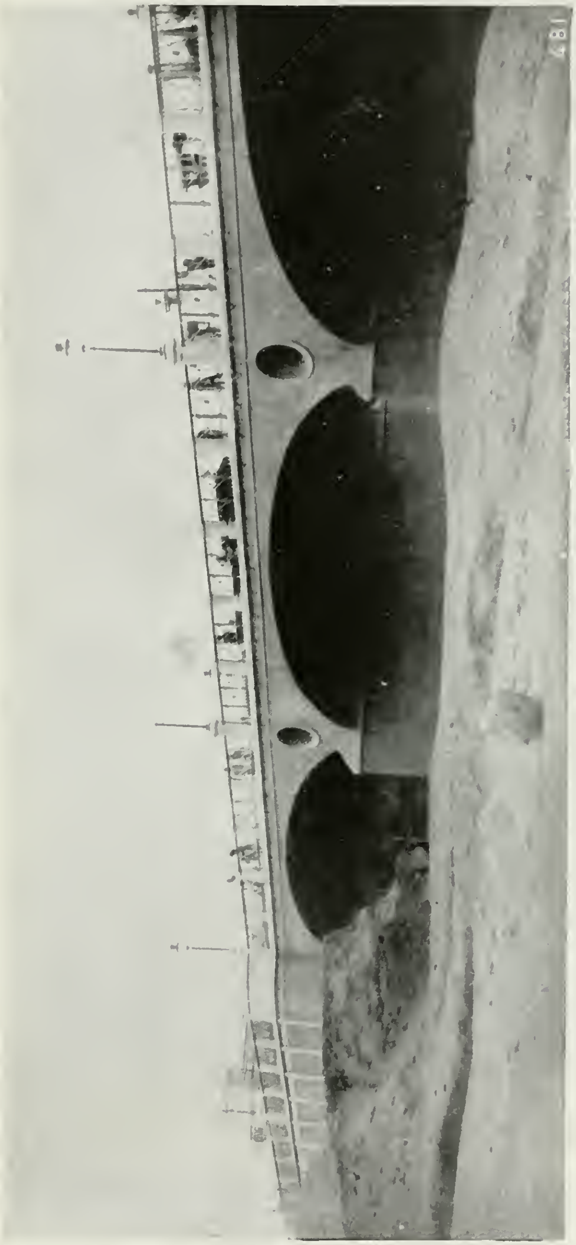


View of C.P. Concrete Piles exposed for inspection.
Foundations for Hovis Mills.

REINFORCED CONCRETE BRIDGE AT HYDERABAD, INDIA.

The Afzalgunj bridge at Hyderabad is the largest reinforced concrete bridge in India. It replaces the portions of the former masonry structure which were swept away during the floods of 1908, and in the design of the new arches a great deal more water-way was obtained.

Messrs. Marsland, Price and Co., constructional engineers, of Bombay, submitted a design for a reinforced concrete bridge which was considered the most satisfactory method of construction, and the entire work was entrusted to this firm.



View of Finished Bridge.

The Afzalgunj Bridge, Hyderabad.

The contract consisted of four elliptical arches of 54-ft. span and 9-ft. rise each, with piers 4 ft. wide built on masonry foundations, and an approach about 200 ft. long.

The piers were constructed of solid reinforced concrete of a 5 : 3 : 1 mixture, suitable bonding rods being left in same to project into the haunches of the arches. The arch rings, which are built of a 4 : 2 : 1 mixture, had a double row of Indented Steel Bar reinforcement placed about 2 in. from the intrados and extrados respectively. These bars, which were 1 in. square, were spaced about 10 in. apart and the two rings connected with binding every 3 ft.

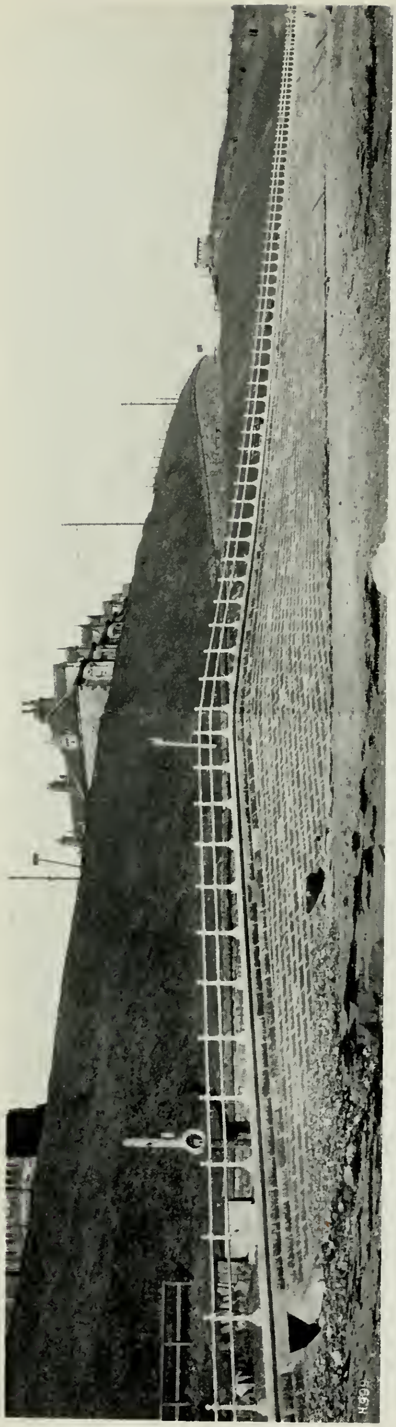
The reinforcement in the extrados of each arch was carried across to join the reinforcement in the extrados of the adjoining arch. No hinges were introduced but work was stopped over the centre of the piers, continuous work being done until each arch was completed.

Each arch ring was 15 in. thick at the crown and about 2 ft. at the haunches, which were afterwards filled in with a weak cement concrete mixture.

The pavement, kerb and gutter were of granolithic. The road of approach has side walls of reinforced concrete "box" construction, each panel being 10 ft. long by 7 ft. wide. The outer wall, 25 ft. high, was 15 in. thick at the bottom and 7 in. at the top, the other three walls of the "box" being 5 in. thick. On the top of these walls the pavement was placed, the hollow inside being filled with lime concrete and stone. The object of this design was to prevent the pavement from settling with the roadway filling, while it gave a cheap form of retaining wall.

The bridge was tested five months after completion. Originally a dead load was prescribed, but later on the authorities considered an 8-ton steam road roller as a moving load would be more practical. This was done with the following results: Deflection with load in centre, $\frac{1}{80}$th in.; vibration chord, $\frac{1}{40}$th in.

Indented steel bars supplied by the Indented Bar and Concrete Engineering Co., of Queen Anne's Chambers, S.W., were used throughout as the reinforcement.



Built of "Winget" Concrete Blocks.

Sea-Wall at Cullercoats, Northumberland.

SEA-WALL AT CULLERCOATS, NORTHUMBERLAND.

The accompanying illustration shows a sea wall constructed with concrete blocks, built by the Tynemouth Corporation at Cullercoats, Northumberland, to prevent coast erosion. Altogether 20,000 32-in. by 12-in. by 9-in. blocks with 2-in. cavities were used. The wall was reinforced with old wire rope passed through the cavities in the blocks the full height and grouted in, thus bonding the whole wall, so as to resist wave action and suction.

The posts and rails in the illustration are of reinforced concrete, the whole being designed and executed by J. F. Smillie, Esq., the Borough Surveyor.

The blocks were made on a "Winget" machine supplied by the (U.K.) "Winget" Concrete Machine Co., Ltd., of Newcastle-on-Tyne.

A REINFORCED CONCRETE COTTON MILL IN BOSTON, U.S A.

The new Maverick Cotton Mills in East Boston, Mass., described here, are about the largest textile mills in the country built entirely of reinforced concrete. This plant was designed and the erection supervised by Lockwood, Greene and Co., mill engineers and architects, of Boston, Mass.

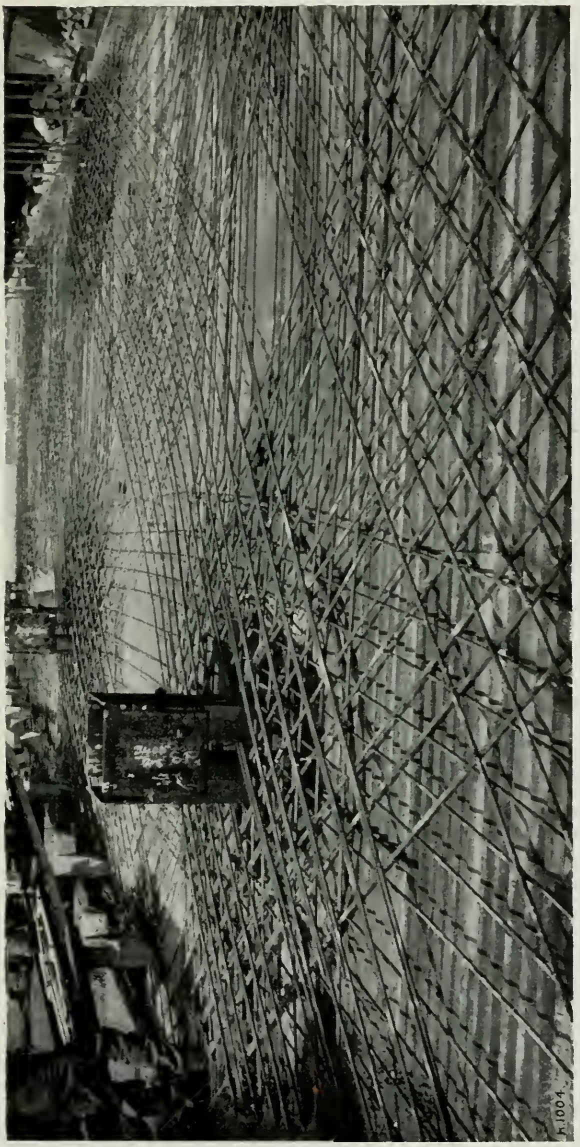
The present buildings consist of the main mill 550 ft. long by 130 ft. wide, two stories in height, with 16 ft. clear from floor to floor; a weave shed 340 ft. by 231 ft., one storey with saw-tooth roof; one end of the main mill joins the weave shed. The power-house is 91 ft. by 62 ft. and adjoins the corner of the weave shed next to the main mill. There is also a small detached office building, 30 ft. by 40 ft., two stories in height.

The engineers were confronted with a number of new and interesting problems in connection with the installation of the textile machinery and the power transmission equipment. In regard to the former it was found that on account of the increased frictional resistance between the concrete floors and the base of the machines that on a large part of the lighter running machinery two or three expansion bolts placed in holes drilled in the concrete floor were all that were necessary. In placing the drawing frame cam tables, which are usually set down into the maple flooring, in this building they were placed on the finished granolithic floor and firmly grouted down, the drawing frames being raised a similar distance above the floor. The fan casing on the pickers caused a little trouble, but was finally secured by using angle cleats and small expansion bolts driven into the concrete floor. An air drill was used for driving the holes in the concrete, which did not entail much more labour than boring holes for lag screws in a wooden floor.

View of Finished Building.
Reinforced Concrete Cotton Mill, Boston, U.S.A.

The entire mill is driven by electric power, the motors being suspended from the ceiling by especially designed frames, with the exception of four 100-h.p. floor motors located in the basement of the weave shed which drive the looms. The proper arrangement of the floor reinforcement, so as not to weaken the floor and not interfere with the locating of the belt holes, required very careful designing on the part of the engineers. The weave shed shafting is supported by ordinary wall hangers bolted to the concrete columns. Vertical strips of wood 5 in. high by 1 in. thick and extending entirely through the columns were embedded in the concrete at the point where the shaft-hangers were to be attached. Holes were bored through these strips and the shaft-hangers bolted directly to the column.

In the spinning mill, where the motors were suspended from the ceiling, horizontal bolt holes were left in the girders at points where the motors were to be attached. Wrought iron plates were securely bolted on either side of the concrete girders, and to these plates were riveted angles to which channels were attached extending from girder to girder. The motors were then slung by heavy bolts from these channels. For the larger motors angle braces were used to stiffen the framework.



View showing Floor Reinforcement in position.

Reinforced Concrete Cotton Mill, Boston, U.S.A.

The actual design of the concrete building contains a number of very interesting features, among which may be counted the heavy girders which support the exterior walls. The problem presented here is that of taking the concentrated load of the columns and transmitting it into uniformly distributed load over the piles. This was accomplished by in a measure designing these girders as ordinary concrete girders uniformly loaded and supported at fixed points and then placing them upside down.

The concrete saw-tooth girders were designed with sufficient stiffness so that no horizontal tie rods from bay to bay were necessary. This gives a clear sweep the entire length of each bay, and, in the absence of any overhead shafting makes a very light and handsome weave shed.

The idea of using reinforced concrete throughout in a textile mill was conceived by Lockwood, Greene and Co., and the entire design of this plant was made by them. They also superintended the erection, the contract for the building being executed by the Hennebique Construction Co., of New York City.

REINFORCED CONCRETE BOWSTRING BRIDGE FOR THE POWELL DUFFRYN COAL CO., SOUTH WALES.

Reinforced concrete is gradually being used to replace steel and masonry works in mines and collieries. As an instance of this the Powell Duffryn Steam Coal Co., Ltd., have recently replaced two steel railway bridges by reinforced concrete bridges on account of the greater economy of this material and its greater resistance to the fumes of sulphur and ammonia, which have no effect upon reinforced concrete. Steel structures subjected to these fumes require constant examination, and there is also a certain amount of yearly expenditure in upkeep for painting and repairs.

The bridges were executed on the "Coignet System" of reinforced concrete by Messrs. Watt Bros., contractors, of Cardiff, under the supervision of the company's engineer, Mr. J. M. Greenhow.

The bridge shown in our illustrations is the larger of the two. This bridge, which spans a river, is very much on the skew. It is calculated to carry two railway lines of ordinary gauge, and for locomotives weighing fifty tons and wagons weighing nineteen tons fully loaded.

Interior View showing Girderless Floor Construction.
Reinforced Concrete Cotton Mill, Boston, U.S.A.

The bridge, which has been designed by Messrs. Edmond Coignet, Ltd., has been provided with two bowstring beams supporting cross beams.

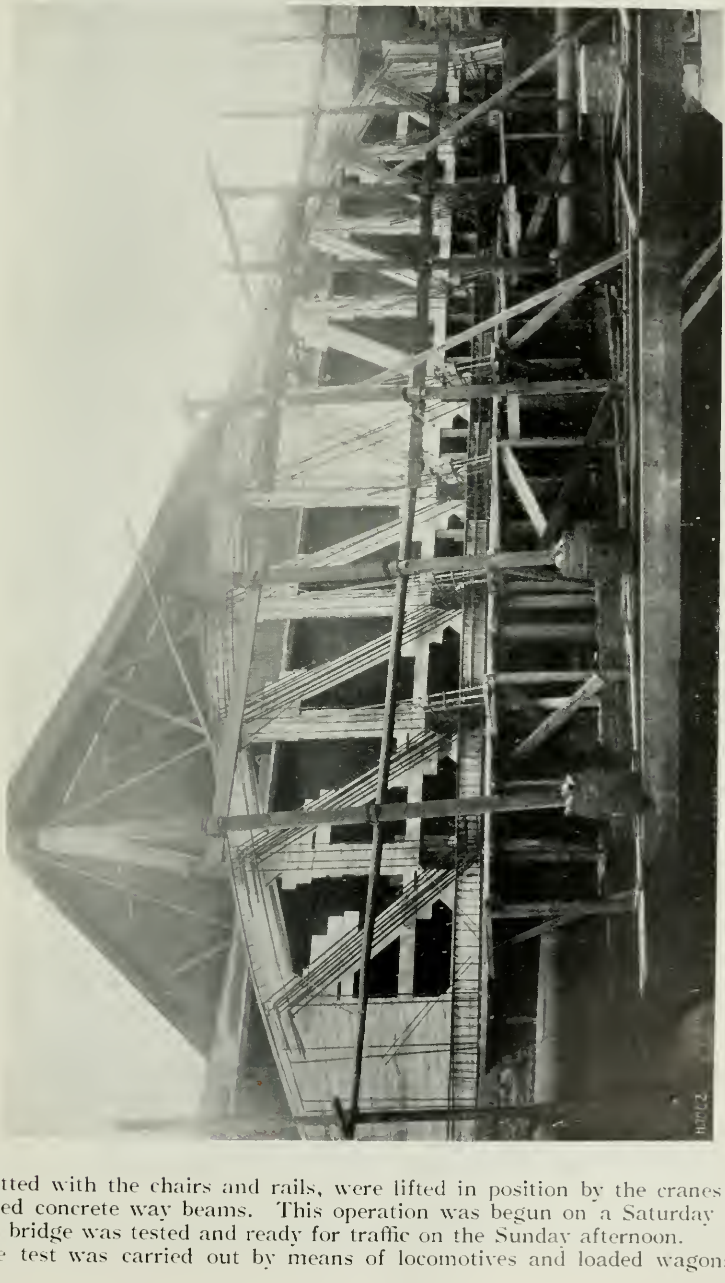
The dimensions are 56 ft. 9 in. in span, with a width of 26 ft. 8 in. in the clear. The height of the bowstring beams is 13 ft. 3 in. over all. The cross beams carrying the way beams have scantlings of 12 in. by 32 in., and the way beams have scantlings of 10 in. by 14 in.

A peculiarity of this bridge is that it was erected around the existing structure and without disturbing the traffic, which is very considerable.

The bowstring beams were first constructed on both sides of the existing steel girders. The filling in of earth of the deck of the existing bridge was then removed, and the reinforced concrete cross beams and way beams were concreted through the openings of the deck of the steel bridge structure.

In order that there should be absolutely no interruption in the work of the colliery, after the reinforced concrete bridge had set sufficiently hard, the deck of the steel bridge was removed by means of travelling cranes, and the wooden way beams, ready fitted with the chairs and rails, were lifted in position by the cranes on to the reinforced concrete way beams. This operation was begun on a Saturday at midday and the bridge was tested and ready for traffic on the Sunday afternoon.

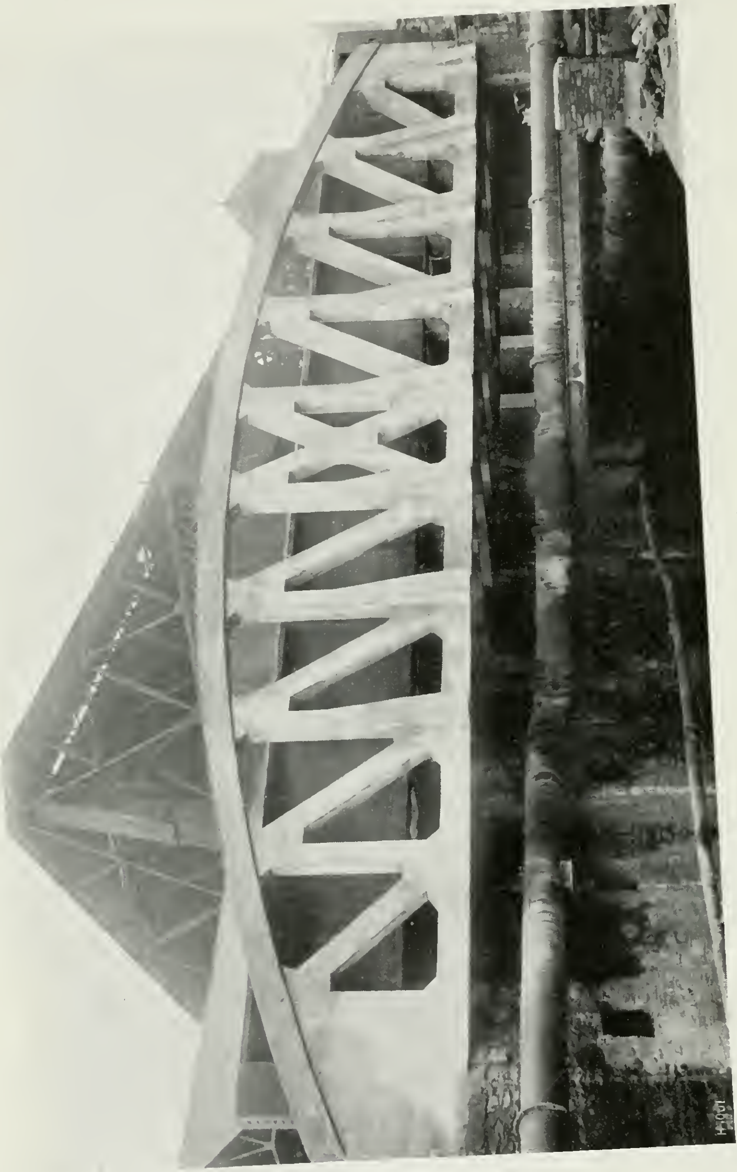
The test was carried out by means of locomotives and loaded wagons, and the



Bridge in Course of Construction.

Reinforced Concrete Bowstring Bridge for the Powell Duffryn Coal Co., South Wales.

elastic deflection measured underneath the bowstring beams did not exceed one-sixteenth of an inch under the most severe conditions of loading. The locomotives and wagons were made to cross the bridge at a certain speed in order to ascertain



View of Finished Bridge.

Reinforced Concrete Bowstring Bridge for the Powell Duffryn Coal Co., South Wales.

the vibration. It was found, however, that the work presented a considerable amount of rigidity owing to its monolithic nature, and the vibration was scarcely perceptible.

It is interesting to note that, although this particular type of bowstring bridge has been used repeatedly on the Continent, the bridge which we have described appears to be the first of this particular type constructed in the United Kingdom.

NEW USES FOR CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented as to new uses to which concrete and reinforced concrete are put, with data as to experience obtained during the experimental stage of such new applications of these materials. The use of reinforced concrete as a substitute for timber in exposed positions is one of the questions of the moment. Railway sleepers, telegraph posts, fence posts, etc., of concrete are being tried. Similarly, efforts are at present being made to prove that reinforced concrete is an excellent substitute for brickwork where structures of great height are required.—ED.

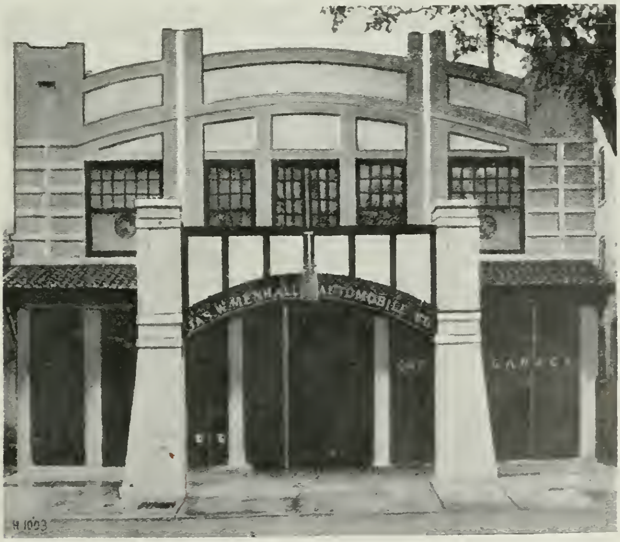
A REINFORCED CONCRETE GARAGE IN BELOIT, WIS., U.S.A.

THE accompanying illustrations, showing the interior and exterior of a fireproof concrete public garage at Beloit, Wis., are good examples of the advisability of using reinforced concrete for buildings where resistance to fire is of primary importance. It is a monolith of reinforced concrete, and has a width of 44 ft., its length being 130 ft., and is two stories high.

It may be stated that the exterior is finished in granite effect, produced by a combination of the whitest cement obtainable, with black and green granite mixed in. The tile embellishments and roofing and the beams are finished in green. The interior is in green and white.

It is of interest to note that the garage occupies the entire ground floor. This floor is unobstructed by pillars or posts, and provides storage space for fifty touring cars or a correspondingly larger number of small cars.

At the left of the front entrance spaces are partitioned off for office and stock

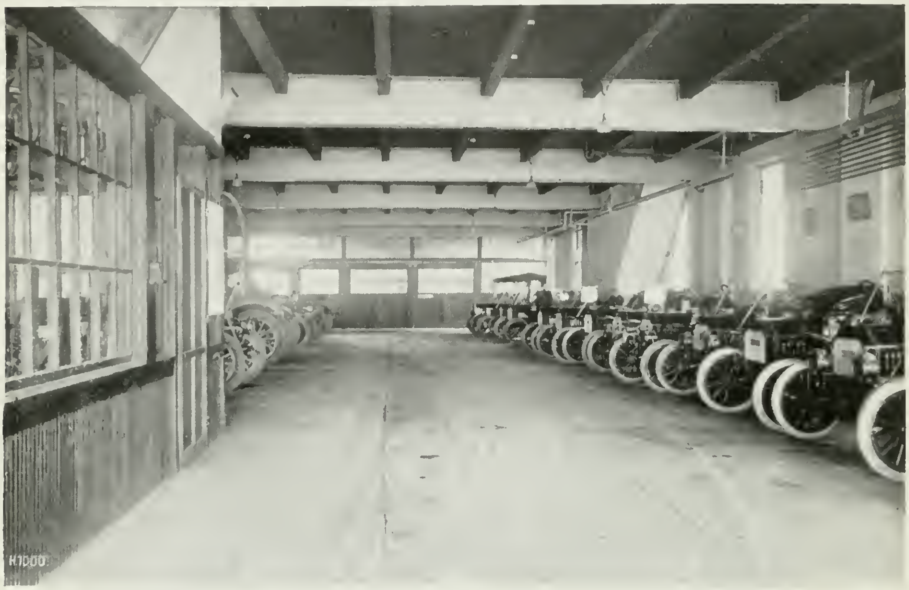


Exterior View of Garage.

REINFORCED CONCRETE GARAGE IN BELOIT, WIS., U.S.A.

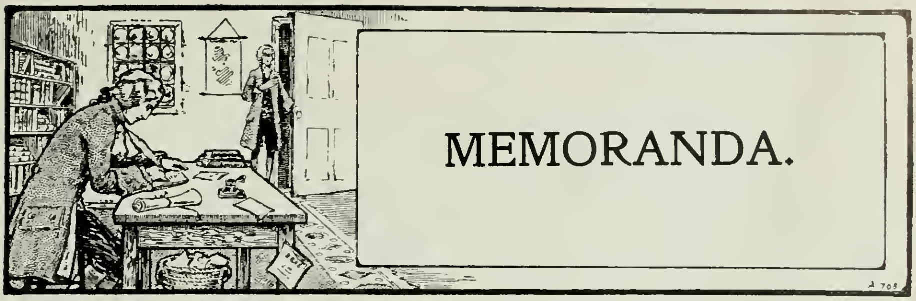
rooms. The administration office is 8 ft. by 12 ft. in size, and the stock or supply room is 12 ft. by 16 ft.

The concrete garage is provided with a repair-shop, which accommodates eight cars at one time, is situated at the rear of the ground floor, partitioned off from the main garage by fireproof sliding doors. The shop occupies a space of 24 ft. by 31 ft. It is equipped with a full complement of machine tools, benches and lathes, and the latest system of gasolene and oil storage has been installed and compressed air for tyre inflation is available at three different points on the main garage floor.



Interior View.

Reinforced Concrete Garage in Beloit, Wis., U.S.A.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

British Standard Definitions of Yield Point and Elastic Limit.—The Main Committee of the Engineering Standards Committee, having been approached with the request that Standard Definitions for the terms "Yield Point" and "Elastic Limit" should be drawn up, remitted the matter to those sectional Committees which had prepared Standard Specifications for Steel and Wrought Iron, with the result that the following definitions have been unanimously agreed upon :—

> *Elastic Limit.*—The Elastic Limit is the point at which the extensions cease to be proportional to the loads. In a stress-diagram, plotted to a large scale, it is the point where the diagram ceases to be a straight line and becomes curved.
>
> *Yield Point.*—The Yield Point is the point where the extension of the bar increases without increasing the load.

A copy of this Report, No. 56, giving further information on the above subjects, is issued gratis (1d. post free) on application to the Engineering Standards Committee, 28 Victoria Street, London, S.W.

Reinforced Concrete Dock for Victoria, B.C.—According to a report of the American Consul at Victoria, B.C., the British Columbian Marine Railway have completed financial arrangements for the construction of a graving-dock at Esquimalt which is to be 1,000 ft. long and will be constructed on similar lines to the one at Southampton. It is proposed to build the dock of reinforced concrete with granite facings, and to employ electric plant for pumping purposes. Work is to be started at once, in order that the dock may be completed before 1915.

A Formula for Reinforced Concrete Slabs.—Rectangular concrete slabs, both freely supported and continuous over supports, have been extensively investigated in connection with the preparation of new Austrian regulations for reinforced concrete, and a revised method of computation is presented. The question is not considered to be conclusively disposed of as yet, but it is believed that an advance is made over earlier methods in that separate computations are provided for the two crossing reinforcements.

In the computation for slab stresses which is now derived, K is taken as the ratio on the sectional area of the reinforcement in one direction parallel to "a," to that of the reinforcement in the other direction, parallel to "b." Then the load is considered as distributed over the slab in the two directions, on the spans "a" and "b" in the relations, respectively : $b^2/(Ka^2+b^2)$ and $Ka^2/(Ka^2+b^2)$.

With slabs continuous over their supports the procedure is similar, but the positive moments are taken to be not less than two-thirds of those in equivalent freely supported slabs.—*Engineering Record.*

Comparative Tests of Cement-Gun Concrete and Hand-applied Concrete.—A series of tests of very interesting character was made recently, according to *Engineering News*, by the engineering firm of Westinghouse, Church, Kerr and Co. for the European Cement-Gun Co., New York City, with a view to determining whether concrete-surface coatings applied by the new air-blast device known as the

"Cement-Gun" are equal or superior to hand-applied coatings. For these tests, sample coatings were made by both hand and cement-gun processes, and tests were carried out for tensile strength, crushing strength, surface permeability, absorption, percentage of voids or porosity, and adhesion to other materials, such as brick, stone, and iron.

The report contains the following conclusions: In all the tests made, the product of the cement gun showed superior to good hand-made product of the same kind. The degree of superiority varied between wide limits. In tensile strength the gun work excelled hand work in every case by amounts ranging from 20 per cent. to 60 per cent. In compressive strength the excellence of the gun work was even more marked, ranging from 20 per cent. to 720 per cent. better than hand work. In the matter of surface permeability the gun work absorbed from $\frac{7}{10}$ths down to $\frac{1}{20}$th as much water per hour per unit of area as the similar hand-made surfaces. As regards absorption of water, the hand-made mortars took up from 1·4 to 5·3 times as much as the gun-made mortars. The percentage of voids of the gun-made product ranged from 52 per cent. to 75 per cent. of that of the hand-made product. The adhesion of the gun-applied mortar was, on the average, 27 per cent. better than that of the hand work.

Filling Holes in Concrete Walls.—Filling holes in concrete walls left by the tie rods of the forms has been done by forcing corks into them and then ramming the holes with grout, according to *The Engineer*. A negro workman at one of the filter beds at New Orleans thoughtlessly jammed the cork of an empty bottle into one of the holes he was supposed to fill with grout. An engineer saw this and shortly afterwards ordered enough corks to fill all the holes, which had previously given more or less trouble from leakage. Corks slightly larger than the holes were used and rammed to the centre of the wall with a rod having a collar several inches from the end and provided with a metal weight arranged so that it would slide along the rod and act as a hammer. The remainder of the hole was then readily grouted.

Church Foundations Consolidated by Reinforced Concrete.—The new church of Notre-Dame at Laeken, the royal suburb of Brussels, was begun about 1853, but building was interrupted ten years later for want of funds, the estimate having been considerably exceeded. The works were not resumed until 1908, and the scaffolding has only been taken down about a month. The spire is 97·5 metres (320 ft.) high, and the two lateral towers are carried up to a greater height than that originally intended, while the three porches project considerably beyond the west front. In these circumstances it is not surprising that settlement threatened the stability of the edifice, notwithstanding additional foundations. The latter were (as advised by M. Lagasse de Locht, now director-general of Ponts et Chaussées) connected inseparably with the old by means of a reinforced concrete sole, supplemented by a 9 ft. vertical curtain for arresting horizontal displacement, the foundation area being increased laterally, especially on the west, so as to afford an admissible coefficient of general bearing stress. At suitable places excavations were made by rock-drilling in the old foundations, for receiving stout iron bars, bedded in cement, connected with the reinforcement members of the new foundations.—*Contract Journal.*

New Baths at Pontefract.—The designs of Mr. A. Nunweek, architect, of Sheffield, have been selected in the recent competition for new baths at Pontefract for the Pontefract Corporation. These are to be carried out in reinforced concrete, and the necessary working drawings are being prepared, for which contractors are then to be asked to estimate.

New Reinforced Concrete Reservoir at Bramley.—The new service reservoir at Bramley, which Messrs. Holme and King, contractors, of Liverpool, are now carrying out for the Leeds City Corporation Waterworks, is being constructed in concrete reinforced throughout with expanded metal, from the designs and under the supervision of the Leeds City Waterworks engineer, Mr. C. G. Henzell.

Horbury Sewerage Scheme.—The Horbury Urban District Council have obtained the approval of the Local Government Board to their new sewerage scheme to be carried out at Horbury, near Wakefield, under their engineer, Mr. J. H. Drew, who proposes adopting reinforced concrete on the Hennebique system in the construction thereof.

New Premises at Keighley.—New premises are to be erected at Keighley for Messrs. Althams from the designs of Mr. F. Mason, architect, of Leeds, who purposes using a system of hollow block concrete floors therein as the system to be adopted in carrying out the work.

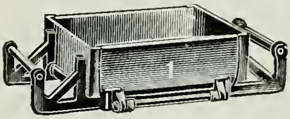
Mill Extensions, Bradford.—Extensive additions are to be made to the Dumb Mill at Bradford, for which specialists in reinforced concrete have been asked to prepare schemes in reinforced concrete and submit estimates for the carrying out of the entire work.

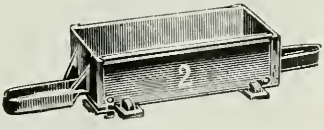
Leeds and Yorkshire Architectural Society.—On the 11th of last month a most interesting and instructive lecture on reinforced concrete was given before the Leeds and Yorkshire Architectural Society at their rooms in the Leeds Institute by Mr. G. C. Workman, of Messrs. Edmond Coignet, Ltd., London, who dealt very fully with the subject of reinforced concrete applied to buildings of all classes. Lantern slides were shown of some very beautiful photographs of works carried out on the Coignet system in Leeds, its surrounding district, and other parts of Great Britain and the Colonies. Some of the work shown had been carried out from the designs and under the supervision of some of the architects present, who formed a very large and most appreciative audience. This was one of the best meetings held by the Society this season. A very full discussion took place, when points of interest were raised as to the importance of centering, and also the subject of tenders for reinforced concrete work.

TRADE NOTICES.

Cement Brick-making Apparatus, "Eureka."—Messrs. H. Elliott and Co., of County Bank Chambers, The Mall, Ealing, London, W., have issued a circular descriptive of their "Eureka" apparatus for making cement bricks.

Cement bricks are easily made, and they can be adapted to any form, shape, size, or mould and used for work a very few days after manufacture. It is stated in the catalogue that one operator can make 1,000 to 1,500 bricks in ten working hours (mixing not included).





The method of using the "Eureka" apparatus is extremely simple. When it is quite full of the mixture remove the apparatus to the place prepared for the bricks, turn over, and then remove the bricks from the apparatus itself by raising the handle, which then allows the apparatus to open automatically. Illustrations of two of the different types of apparatus are shown above.

Full particulars and catalogues will be sent on application to Messrs. Elliott at the above address.

"Universal" Joist Steel Piling and the Reconstruction of Boulter's Lock.—It will interest our readers to know that "Universal" joist steel sheet piling has been ordered by the engineer for the Thames Conservancy for the reconstruction of Boulter's Lock.

United States Steel Sheet Piling.—We have received from the United States Steel Products Company, whose London offices are at 36-38 New Broad Street, E.C., a booklet giving particulars of their method of piling.

The advantages of steel sheet over wooden piling are well known. It is much stronger and tighter than wood piling, and in extended trench work and multiple piers its availability for re-use makes it cheaper than wood. Its use in coffer-dams reduces the amount of bracing required, and thus effects a saving in the cost of timber.

The United States sheet piling is a simple, plain, rolled section. It is made of medium steel in accordance with the specifications of the Association of American Steel Manufacturers, and is ready for immediate use as it comes from the mill.

OUR PILE-DRIVING PLANTS ARE THE LATEST ON THE MARKET ◆ ARE USED BY AND GIVE SATISFACTION TO THE LEADING CONTRACTORS AT HOME AND ABROAD ◆ RELIANCE & EFFICIENCY ARE OBTAINED BY USING OUR MACHINES

Illustration shows Overhanging Frame with Boiler and Winch, for Buenos Ayres
(View in Works Yard)

We quote for any kind of pile-driving machine; drawings and prices on application. Send for particulars of our latest type of hammer, to drive with either compressed air or steam.

THE BRITISH STEEL PILING CO.
DOCK HOUSE, BILLITER STREET
LONDON, E.C.

TELEPHONE—
1414 AVENUE.

TELEGRAMS—
"GRAMERCY, LONDON."

Each piece is complete in itself, interlocking, and all pieces are interchangeable with one another. There are no auxiliary parts, which constitutes a difference from other pilings on the market.

Contact between the head and the socket is made by points and not by surfaces, so that wedging action is prevented and the maximum of strength to resist forces in both lateral and longitudinal directions is secured.

Among the other advantages claimed for this piling are that, owing to the small amount of metal in actual contact at the point of interlock, free and easy driving, irrespective of the length of the pile or the composition of the soil, is assured. The amount of play at the same point has the further advantage of enabling the contractor to form a circle having a stipulated minimum diameter, without resorting to the somewhat expensive method of bending the web.

If the river bed is composed very largely of sand and clay the piling will automatically pack itself when being driven, but if absolute watertightness is required it is permissible and practicable to insert a timber packing strip within the interlock. This packing strip should be made of some tough grained wood, such as spruce or fir, which swells considerably when immersed in water. The strip should be rectangular in section, so that contact can be made by points rather than by surfaces. We are informed that this packing strip, contrary to expectations, usually acts as a lubricant than otherwise when driving, and, if anything, facilitates penetration.

The material can be supplied in almost any length ranging from 10 ft. to 60 ft. Full particulars are given in the booklet of the method of driving the piles, and there are illustrations of piling caps to suit either the American or British type of piling frame.

The "Moseley" System of Reinforced Concrete Construction.— The Moseley system of reinforced concrete, for which provisional patents have now been taken out, is a new system that is being put upon the market for the formation of reinforced concrete troughs, for the quick erection of walls, and all other similar works in concrete. The system consists of cast reinforced concrete troughs with perforated bottoms laid in courses the same as brick or stone work, and filled in with *in situ* concrete course by course and then grouted together with liquid cement, which penetrates the perforated bottoms, so becoming one homogeneous mass. The joints are all strengthened and tied together by metal reinforcement as the work proceeds. Particulars may be obtained from the Moseley Construction Co., whose temporary offices are 6A, Grosvenor Chambers, Deansgate, Manchester.

PUBLICATIONS RECEIVED OR IN COURSE OF PREPARATION.

Lockwood's Builders and Contractors' Price Book, 1912. — *Published by Crosby, Lockwood and Son, Stationers' Hall Court, Ludgate Hill, London, E.C. Price 4s.*

This price book, which is now established as a standard work of its kind, gives in its 1912 edition in the same complete and efficient manner as in former volumes the prices of the different materials and work in the building and allied trades, these having been brought thoroughly up to date. The wages table has also undergone entire revision and enlargement.

The appendices contain tables of weights, areas, valuation of leases and estates, marks and qualities of woods, stamp duties, and legal notes. In the supplement will be found among others a copy of the London Building Acts 1894 to 1908, the L.C.C. (General Powers Act) 1908, and the text of the L.C.C. By-laws and the Regulations of various public health departments. The building contract and schedule of conditions of the R.I.B.A. will also be found a useful addition.

Hennebique Ferro-Concrete. — *Published by Messrs. L. G. Mouchel and Partners Ltd., 38 Victoria Street, London, S.W.*

This pamphlet, which gives a list of the works executed in the United Kingdom on the Hennebique system from 1897 to 1911, has been compiled for the convenience of engineers and architects desiring to inspect examples of reinforced concrete construction. The pamphlet gives in each case the locality in which the building is situated, the engineer or architect who designed it, as well as particulars of the structure. It should be found very useful for the object for which it is intended.

Catalogue of Technical Books.—Messrs. W. and G. Foyle, of 135 Charing Cross Road, have sent us a copy of their latest catalogue of technical books, in which is to be found many books on engineering in all its branches, architecture, and chemistry, as well as other technical subjects. Second-hand copies of nearly all these books can be supplied by Messrs. Foyle at about half the published price.

Applied Chemistry.—Sir Edward Thorpe's well-known "Dictionary of Applied Chemistry" is being revised and enlarged, and Messrs. Longmans and Co. announce that the new edition will be issued in five volumes. The first volume will be ready in a few days, and Volume II. early in the summer. It is hoped that the work will be completed within two years.

CONTRACTS.

Messrs. Edmond Coignet, of 20 Victoria Street, S.W., have recently secured the following Contracts:—

New offices for the Metropolitan Railway at Baker Street Station. Engineer, Mr. W. Willox, M.Inst.C.E. This contract, which is for the supply of technical advice and working drawings, was obtained in competition amongst the various well-known systems.

Large warehouse for Messrs. Pilkington Bros., Ltd., of St. Helens. Architect, Mr. Isaac Taylor, F.R.I.B.A. The Coignet system was specified exclusively for this work. The contract for the erection of the building was obtained by Messrs. William Cowlin & Son, of Bristol.

Extensive additions for the British Motor Cab Co., Ltd. Contractors, Messrs. Leslie & Co., Ltd.

New administration buildings at Para. Engineer, Mr. J. W. Kitchin. The Coignet system was specified exclusively for this work, the contractors being Messrs. William Cowlin & Son.

Roof for Church of St. Mary the Virgin, Plymouth. Architects, Messrs. Nicholson & Corlette. The Coignet system was specified exclusively for this work. Contractors, Messrs. William Cowlin & Son.

Floor for extensions to Messrs. C. N. Kidd & Son's brewery at Dartford. Contractors, Messrs. W. F. Blay Ltd.

Floor for school at Alfreton. Contractors, Messrs. Evans Bros.

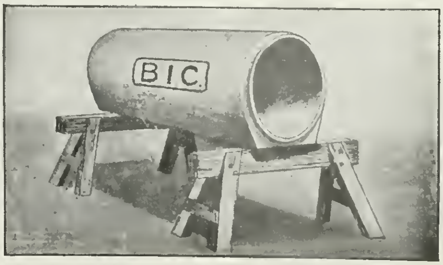
Extension to flooring at Bailiffe Bridge for Messrs. T. F. Firth & Sons, Ltd. Contractors, Messrs. Henry Atkinson & Sons, Ltd.

"BIC" BRITISH "BIC"
IMPROVED CONSTRUCTION CO., LD.

TELEPHONE: 4067 VICTORIA. TELEGRAMS: "BICONCRETE, LONDON."
47 VICTORIA STREET, WESTMINSTER.

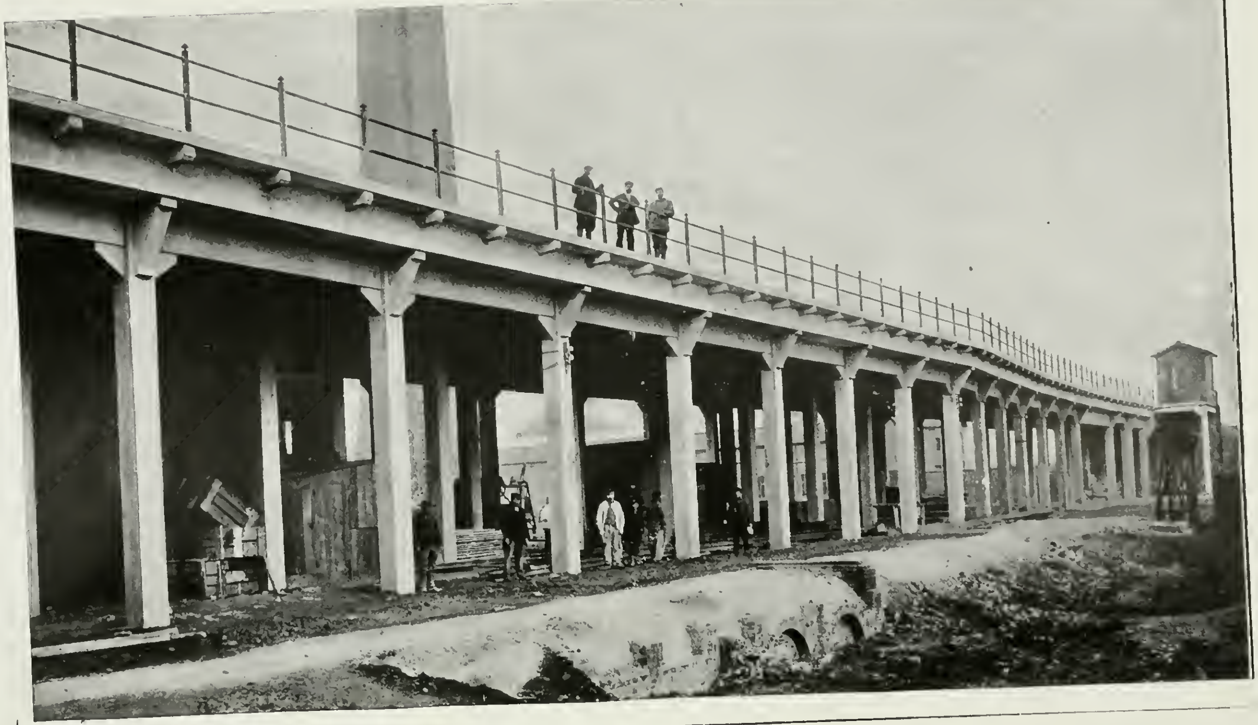
Flat-based Concrete Pipes
Three feet long to Nine feet long.

Breeze Partitions
Reinforced Sleepers
Telegraph Poles



Porous Pipes
Concrete Beams
Hollow Floors

SPECIALITY:—INVENTORS' DESIGNS CARRIED OUT TO ORDER.



View of a portion of the Viaduct. (*See description on page 228.*)

Reinforced Concrete Viaduct at the Gennevilliers Gas Works, near Paris.

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume VII. No. 3. LONDON, MARCH, 1912.

EDITORIAL NOTES.

THE EXECUTION OF REINFORCED CONCRETE WORK.

COMPLAINTS are frequently uttered about the incompetence of foremen and workmen in the execution of reinforced concrete work. This is certainly a serious matter, for the majority of mishaps that have occurred have generally been attributed, whatever may have been contributory causes, primarily to the ignorance of the contractors' employees. Now, the contractor is not exactly to blame; he picks out the best material available, but that material is, in the majority of cases, inefficient.

Reinforced concrete is, of course, a new method of construction to the greater portion of the men engaged in the building trades, and consequently the foremen or superintendents neither understand nor appreciate the nature and difficulties of reinforced concrete construction. The drawings, too, are strange and not easily interpretable, because they are rather different from the ordinary run of drawings of building work. The foremen generally do their best, though they do not understand the drawings, but there is also gross carelessness with many.

The ordinary ideas of concrete in the building trade is that it is a sort of mud-pudding material, which is thrown into holes to fill up and give a smooth base upon which to put other materials. Even when it comes to floors, the average building employee has been used to steel joist floors with breeze or a clinker concrete filling of a very inferior quality. Our foremen are often, too, taken from trades which in the past were totally ignorant of, or inadequately equipped with, a knowledge of the strength of structural members. Carpenters and joiners had knowledge of timber joists and beams, but not of the calculation of the stresses induced in them by loading. They were mostly unable to calculate the strength of even a rectangular beam except by rules of thumb. Masons are even more insufficiently equipped, because, although they have some ideas of the properties of an arch, beam-construction in stone is not in much use nowadays. So that a foreman picked from these two chief branches of the building trade has the haziest notion of what is essential or non-essential to the strength of structural members. The other trades do not so frequently supply foremen.

The various polytechnics and schools undertaking instruction of the building trades, in their courses teach building construction to most of the craftsmen, but that teaching is often inadequate, and emphasis is not generally given to the little science needed for the understanding of the main principles of strength of materials, too much attention being devoted to matters of really secondary importance, such as details and handicraft.

No doubt in time matters will right themselves, but it will be slow work, and it would be well if the whole of the trade concentrated its attention on forcing the foreman element to understand something of the principles of reinforced concrete, and hurried up the process of instilling knowledge into craftsmen. Unfortunately the foremen who have had experience, and clerks of works also, are often disinclined to study. They think that after they have attained to a position of such responsibility there is no need for them to go to school again; they wait for practice to inform them upon any novelties. They are really foolish in their own interests, because a thoroughly capable man able to do first-class work in reinforced concrete is able to command good wages and continuity of employment.

However, there is another aspect of the matter which must be faced. The execution of reinforced concrete work has been handled by a large number of building firms, but the greater proportion of the work has been carried out by a comparative few, who have become known somewhat as specialist contractors for reinforced concrete work, and it is rightly and properly thought that such firms are best able to carry out such work with safety and economy. These firms have chosen their foremen and superintendents with care, and they possess some of the best material available, but there is an element of danger, noticeable particularly in work in the United States, and it would be well for us to avoid appearing here, and that is, that in the growth of reinforced concrete construction and the increased business obtained by the specialist contractors they may find insuperable difficulties in obtaining competent foremen. It is only natural that they should not wish to let jobs go by, and they may be led to take them and endeavour to carry them out with the best staff they can secure.

Now, the best foreman talent has been already secured, and is in regular employment with general building contractors, and therefore the sources of supply of such talent to the specialist contractors are somewhat restricted, so there is a tendency for him to draw from sources which are not quite the best, however poor even that may be. He has to take men on trial who have had little or no previous experience of reinforced concrete, and who cannot therefore furnish testimonials as to their ability. In America the result has been that the firms that had the most amount of work were sometimes found to be doing the worst work, so that there is this danger, that the firm which builds up the best reputation may suddenly find as a consequence that they are getting so much work that they quickly drop from the first place. Not being able to obtain adequate superintendence, they begin to turn out inferior work, and may lose the reputation which has taken them years to build up, and they can never regain it. It is a very regrettable thing that there should be such danger of the specialist firm, by the very development of its superior work, losing its ability to secure efficient workmanship, and doing inferior work even to what would be done by general contractors.

We see, therefore, that there is a limit to be set to the operations of reinforced concrete contractors, if they wish to continue to maintain their reputations. That limit is the amount of efficient labour that can be obtained. Also it becomes increasingly obvious that while the specialist contractor will probably remain, the execution of reinforced concrete work by others will become more general, because there is sufficient room for superior talent in the carrying out of the work that the ordinary building contractor will in time become more and more accustomed to its execution, and the majority of the simple jobs will be carried out by him. That will probably result in a gradual change in the method of designing work, and the specialist designers will develop from being so closely connected with trade into more or less consulting engineers, then the trouble of excessive duplication of labour in designs will automatically cease also.

The great thing, therefore, for specialist contractors to strive against is the insidious manner in which by the growth of their businesses they may devolve from good to inferior and bad work. It is hard to set oneself against taking on work when it is offered, and to continue to have a small output and make a regular profit year after year when it appears so easy to increase; but that will be the most effectual procedure in the end. They should make up their minds to carry out only a certain amount of work which they have the staff capable of doing, and see that they get proper prices. If they do that they will not need to complain of excessive cutting of prices, and they will be able to go on in a profitable business, whereas the opposite road leads to continual striving to increase the output and cheapen the production so as to enable undercutting. This tends in a short while surely to reduce the profits for everyone, and to bring the rates down to such a point that few can do good work and make a profit. It has been proved in general contracting work that such a policy is a foolish one, because a few firms are quite unable in the building and engineering industries to freeze out their rivals and secure a monopoly; a combine is quite impossible in these trades. Therefore everyone in the end loses by such undercutting. If the specialist contractors are wise they will be careful not to undercut, but to maintain a fair price for good work, or refuse to do work too cheaply with its natural degeneration into shoddy work. In the end the public will appreciate that good work must be paid for, and, fortunately, the engineering and architectural professions are pretty well aware of it; at any rate, the leading members know what is good work, and they are prepared to back the firms that do it.

As regards the general outlook, of course, it is an advantage if the cost of reinforced concrete work can be reduced, because it will lead to increasing use of this method of construction, and be to the advantage of the public, but that reduction in cost must be obtained in a true spirit of economy by the reduction in the amount of labour and materials, due to the provision of superior methods and materials, and of more scientific design and execution, and not by undercutting, shoddy workmanship, and poor materials.

REINFORCED CONCRETE AND GAS WORKS.

A CONSIDERABLE time back we published particulars of some large new gasworks at Copenhagen, in the construction of which reinforced concrete had been utilised in a comprehensive manner.

We have had an opportunity recently of studying the application of this material at another great modern gasworks—namely, that of Genevilliers, near Paris—and with these two examples before us, we have come to the conclusion that both for practical and economic reasons no material can possibly lend itself better to this particular class of constructional work than reinforced concrete.

We therefore take this opportunity of pointing out to our many engineer leaders who hold municipal appointments that the application of reinforced concrete to this particular form of municipal undertaking must be greatly in the interests of the communities they serve.

Where such municipal gasworks are erected or extended with money obtained under loan on the certificate of the Local Government Board, we realise that, owing to the conservative attitude of that body *versus* this modern constructional material, difficulties are likely to arise as to the loan period, but we think that these difficulties would probably not be so great with work above ground as in the case of gasworks, as perhaps with structures below ground and subject to the continual application of water.

In any case, we think that this subject is one that requires the attention of engineers and managers controlling gasworks as a purely economical and practical problem, and that the Institution of Gas Engineers would be well advised to take up the matter seriously, and if possible to obtain some definite assurance from the Local Government Board as to the principles they are likely to apply on the question of the loan period where reinforced concrete is used in gasworks.

We would not deal with this matter so emphatically were it not for the fact that very large sums of money are being annually wasted by the ratepayers of the United Kingdom in utilising unnecessarily expensive methods of building construction, where reinforced concrete would be equally or more economical, and it behoves every public authority to put a stop to such unnecessary waste.

ADDITIONS TO THE HOTEL CECIL, STRAND.

The alterations to the Hotel Cecil entailed some very interesting features from the constructional point of view, and the account given below, which was prepared for us by Mr. Albert Lakeman, Hon.-Medallist Construction, is worthy of attention.—ED.

Extensive alterations and additions have been practically completed at the above from the designs of Mr. E. Keynes Purchase, F.R.I.B.A., and some interesting construction has been executed owing to the difficulty of dealing with the existing work. The new work includes the formation of two large palm courts, which occupy about two-thirds of the old courtyard adjoining the Strand, and in addition new manager's offices, service rooms and cloak rooms have been constructed.

The general scheme is illustrated in the plan shown in *Fig.* 2, which up the east and west blocks, and this is built in Portland stone with Ionic columns and carved pediment, and five large openings are formed for entry to the grand vestibule, banqueting-rooms and luggage room. The grand vestibule measures 48 ft. by 24 ft. and has a height of 16 ft., and from this access is had to the lower palm court, the restaurant corridor, and a wide covered way leading to the south block of the hotel. The lower palm court is a very fine room, measuring 90 ft. long by 45 ft. and 32 ft. high, and at right angles to this and adjacent to the south block is the upper palm court, which is 120 ft. long, with an average width of 30 ft. The latter has the floor level 4 ft. above the lower palm court, and the two are connected with three large arched openings with a wide staircase in the central one.

The general scheme is illustrated in the plan shown in *Fig.* 2, which also gives the relation of the new work to the old. The principal entrance to the hotel prior to the commencement of the work was situated in the centre of the north wall of the south block, and as this was enclosed by the new construction it became necessary to form a temporary entrance and reception-room at the lower end of the courtyard next the Strand. This temporary reception-room had a maximum length of 36 ft. 6 in. and a maximum width of 20 ft., with a height of about 15 ft., and the walls were constructed entirely of hollow pumice concrete slabs 4 in. thick plastered on the inside and rendered on the outside with fine washed sand and Portland cement jointed to imitate stone. After this room was constructed a hoarding was erected between the east and west blocks, and the whole of the upper part of the courtyard was given up to the building operations.

The foundations for the new palm courts presented some little difficulty,

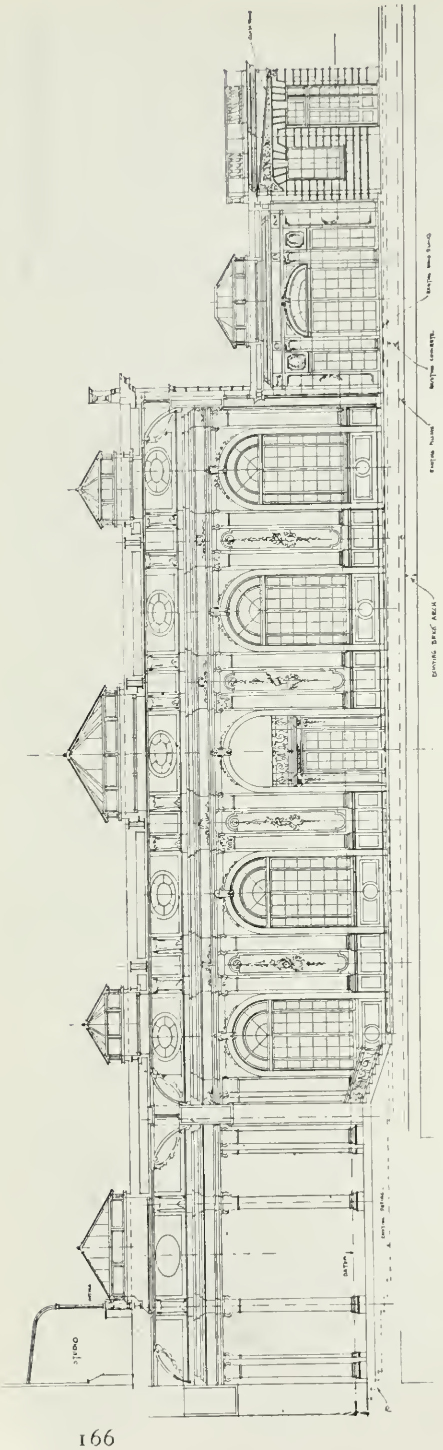


Fig. 1. Longitudinal Section.
Additions to the Hotel Cecil.

owing to the fact that the space below the courtyard is occupied by various departments of the hotel entered from the Embankment level, which is about 30 ft. below the Strand level at this point. These departments comprise the laundry, engine-rooms, boiler-rooms, bakehouses, carpenters' shops, and other similar offices, and it meant bringing heavy loads over these rooms in such a manner as would not endanger the substructure, and at the same time allow the business of the hotel to proceed. The plan in *Fig.* 5 illustrates the line of the walls below the courtyard, and also the relative position of the new work above. These sub-buildings are arranged with a central roadway 20 ft. wide, which extends right up to the underside of the courtyard and is finished at the top with an 18 in. brick arch across the span of 20 ft. This arch has its abutments on the side walls, which are nearly 3 ft. thick, and at right angles to these walls others of a similar thickness run back on either side to the face of the east and west blocks respectively, forming large compartments about 22 ft. wide, and these are also finished at the top with arches 18 in. thick at right angles to the main arch. These compartments are divided into two heights by an intermediate floor.

The weights from the new superstructure were concentrated at certain definite points by the stanchions, and it was obviously insufficient to place these loads on the arches themselves, and the only satisfactory method was that of transmitting the weight to the lower walls direct and distributing it over such an area so as not to exceed the

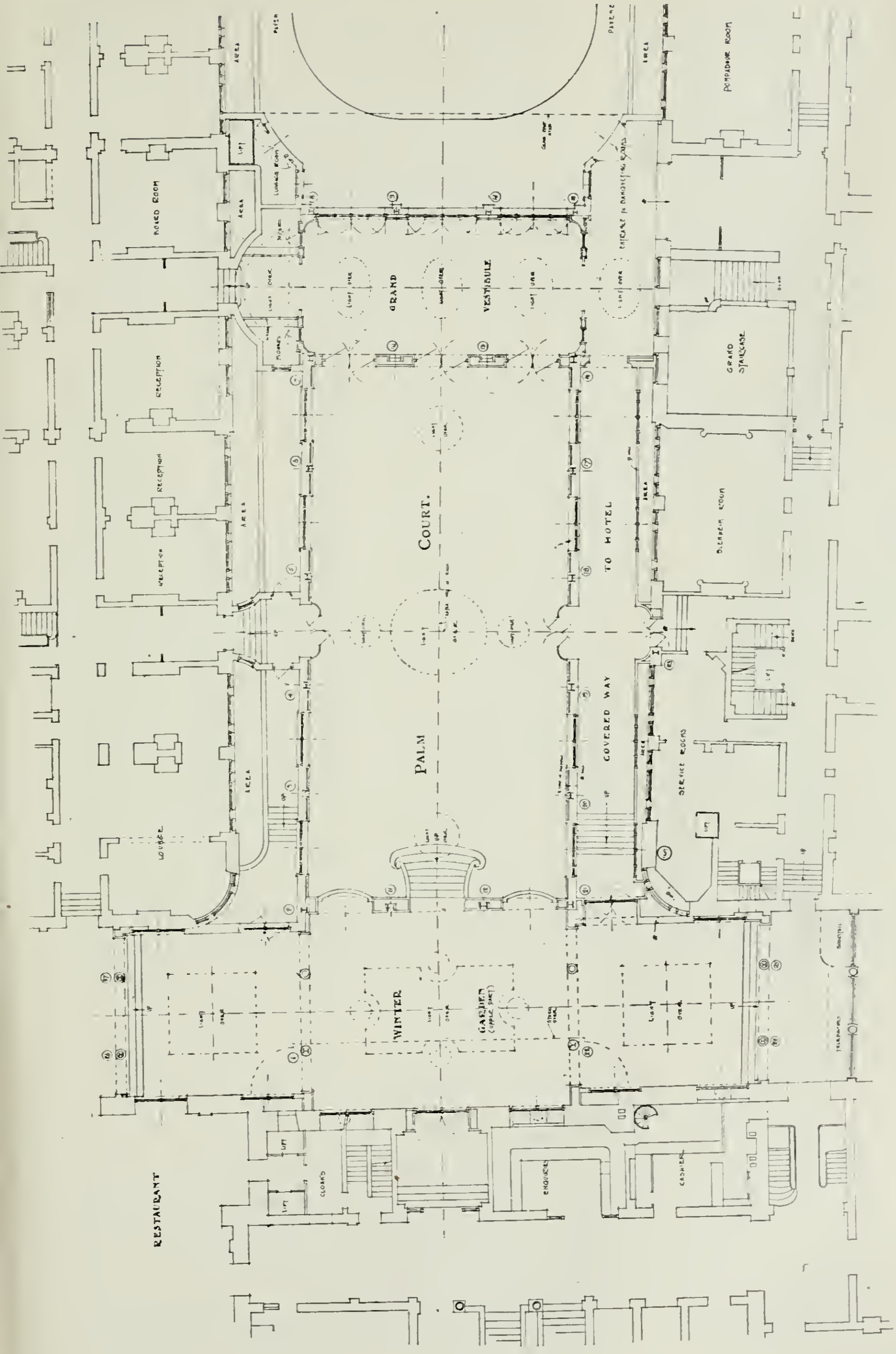


Fig. 2. Plan showing the new Palm Courts.
Additions to the Hotel Cecil.

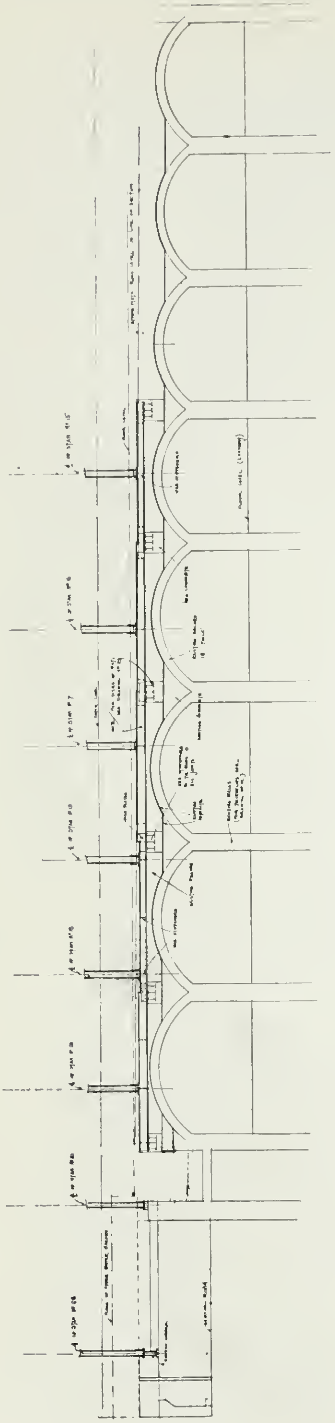


Fig. 3. Section through Side Arches.
Additions to the Hotel Cecil.

safe load on the brickwork. This latter was found to be hard brickwork and in good condition, and the great thickness, together with the good condition, allowed this to be done.

A system of grillage beams was adopted consisting of a top layer of three rolled-steel joists parallel to the main walls of the palm court, which actually carried the stanchion and wall loads, and these were supported at the ends by a bottom layer of three similar joists placed at right angles and resting on a thick bed of concrete placed between the haunches of the side arches and directly over the walls beneath. As will be seen in the section illustrated in *Fig.* 3, the top layer beams were kept quite clear of the crowns of the arches and no pressure was allowed to come upon these. Web stiffeners were provided in the top layer beams and bolts and distance pieces were put to the bottom layer beams and the whole of the steelwork was enveloped in cement concrete, having a minimum covering of 2 in. The size of the steel beams varied from 14 in. by 6 in. to 24 in. by 7½ in., and the length of the bottom layer was 16 ft. in some cases in order to distribute the load over a sufficient area. The greater portion of the foundation work was executed in sections, while the courtyard was kept open for traffic, and this rendered the work more difficult.

In the superstructure itself brick walls 18 in. thick were employed, with a concrete slab lining 4 in. thick placed on the inside of same with a 2-in. cavity between, the object being to present a perfectly dry surface on the interior for the application of the fibrous plaster decoration, while the loads were carried by compound steel stanchions placed generally at 19-ft. centres. These stanchions carried the ends of the main roof girders, which have a span of 46 ft. 6 in. across the full width of the palm court.

The stanchions were composed of one 12-in. by 6-in. by 44-lb. R.S.J. and two

12-in. by ½-in. plates, the total height being over 33 ft., while the main roof girders were built up with two 24-in. by 7½-in. R.S.J. and four 16-in. by ½-in. plates, this giving a girder only 2 ft. 2 in. deep, which was an important consideration, as a flush ceiling was required in the courts and a very deep girder would have entailed a certain loss in height. These girders weighed over 7 tons each, and as the space for working was very limited and no crane was available their great length necessitated some care in raising. The roof itself was constructed with hollow Siegwart beams, as these obviated the necessity for any centering, and, in addition, were brought to the site quite dry, and thus saved a great deal of moisture from being introduced into the building. These beams had an average span of 10 ft. with a thickness of 4½ in., and they were carried by 12-in. by 6-in. R.S.J. placed at right angles to the main compound roof girders previously described and framed into them. The beams were required to carry an external load of 112 lb. per ft. super, and a beam tested on the site was loaded up to 784 lb. per ft. super before any signs of failure occurred, and at this point a small shear crack occurred at one end. Fine concrete screeding was placed on the top of the beams when in position and the whole covered with asphalt.

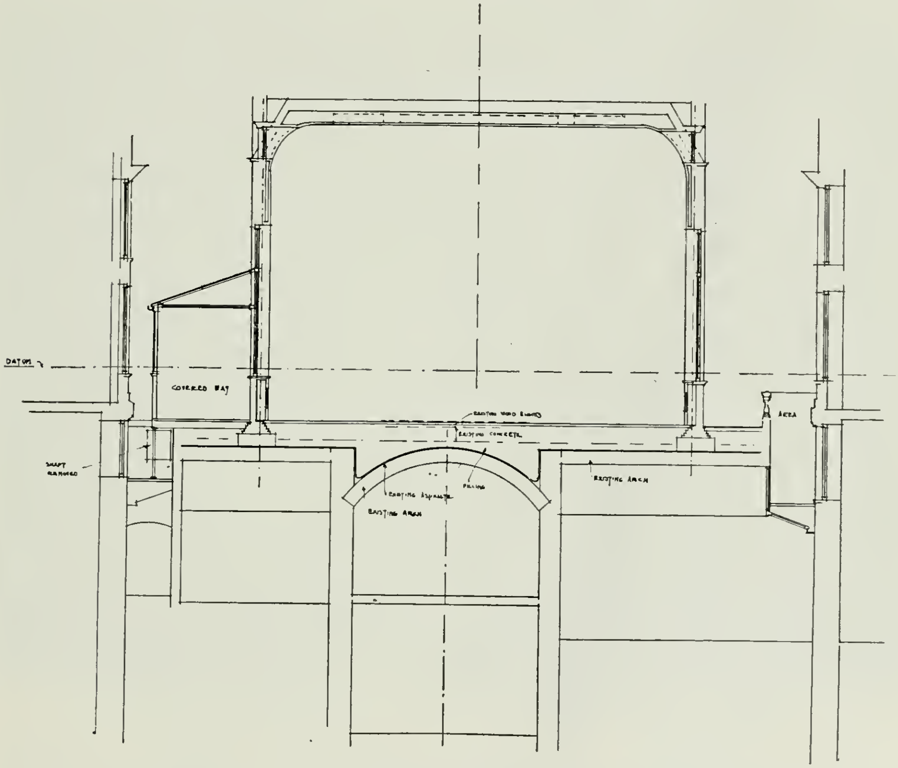


Fig. 4. Cross Section.
Additions to the Hotel Cecil.

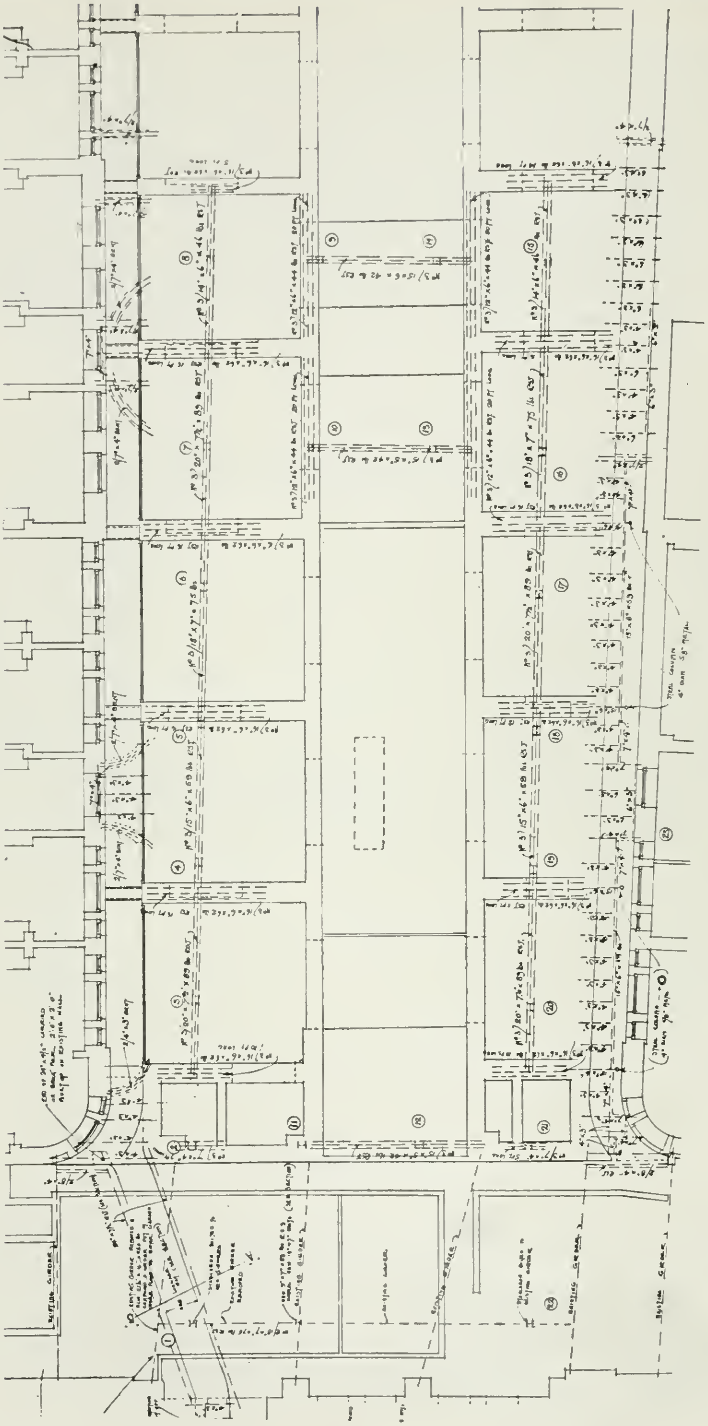


Fig. 5. Plan showing Steelwork in Foundations and Walls Below.

ADDITIONS TO THE HOTEL CECIL.

The most difficult portion of the superstructure work was that in the centre of the upper palm court next the south block. A large stone porch over 60 ft. long and 30 ft. high existed at this point and carried on the top of same was a photographic studio. The new scheme necessitated the removal of the porch while the studio was allowed to remain, and an elaborate system of shoring was required to uphold this during the removal of the stonework and the insertion of two main girders which passed right through the studio floor to bear on the main wall of the south block. The two extreme corners of the studio are now carried on compound steel stanchions which have their base plates resting on the large compound steel girders which originally carried the end stone piers of the porch. The interior of the palm courts will be finished with fibrous plaster decoration in the style of Louis XIV., and the floor will be polished Austrian wainscot oak.

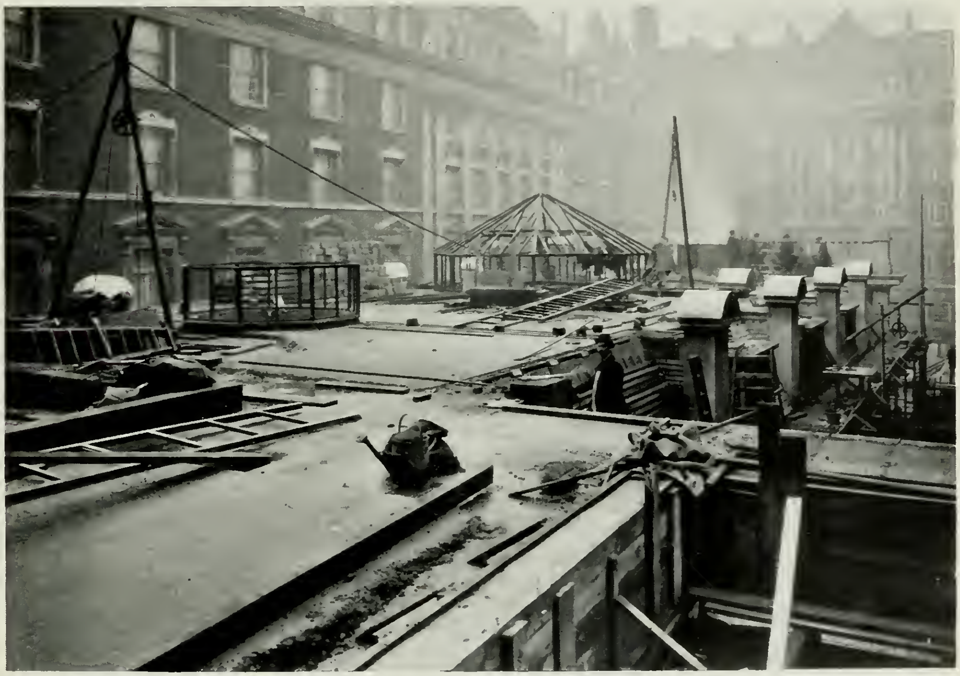


Fig. 6. Roof of Palm Courts.
Additions to the Hotel Cecil.

Of the alterations in the existing buildings, the most interesting, from a constructional point of view, was that of the formation of new service rooms on the ground floor of the south block for use in connection with the restaurant. The building of the upper palm court necessitated the removal of the old service rooms, and it was decided to convert three rooms which were situated on the east side of the restaurant kitchen. To provide communication between the kitchen and the new service rooms it was decided to cut an opening about 10 ft. wide and 9 ft. high in an 18-in. brick wall which divided the two and extended for the full height of the south block, a distance of over 100 ft. from the ground floor level. Upon the removal of some marble lining at this point it was found that a serious crack existed in the brickwork, apparently due to some kind of settlement. A large opening occurred in the wall in the American bar situated on the floor below, and upon calculation it was found that an enormous load was carried over this opening due to several weights brought on to the wall at higher points in the building, and fears were entertained that the existing girders over this opening were insufficient and that deflection had occurred. Upon the exposure of the work at the head of the opening it was found that the whole load was carried by two 16-in. by 6-in. R.S.J., which were theoretically absolutely unsafe.

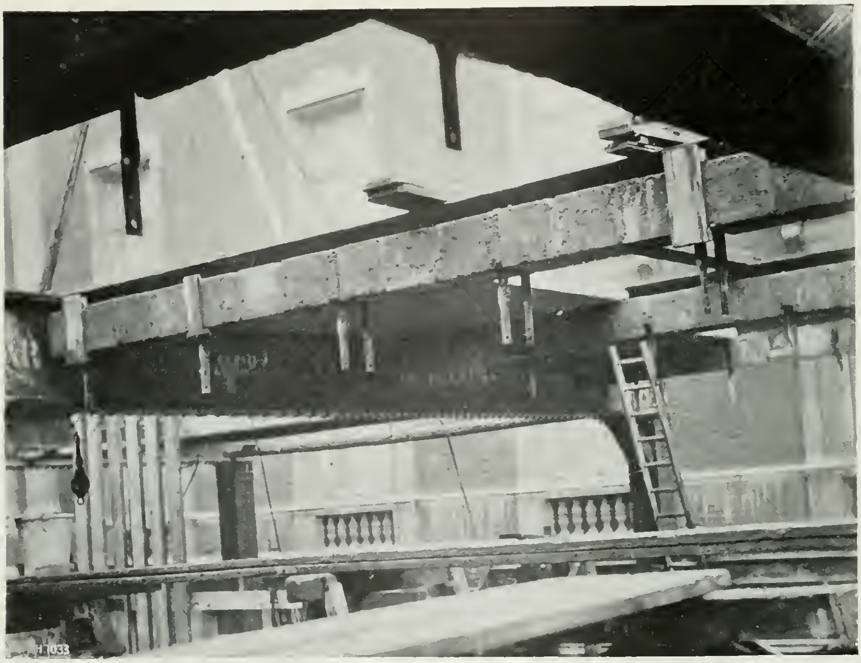


Fig. 7. Beam Casing and Hangers.
ADDITIONS TO THE HOTEL CECIL.

The position of the proposed new opening between the new service room and the kitchen on the floor above was such that a great amount of extra weight would be diverted in such a manner as to bring it directly over this opening in the American bar, and consequently it was unsafe to proceed until some method of strengthening the latter opening was adopted. A steel frame was therefore designed to place inside the opening, consisting of two compound girders 20 in. deep at the head, resting on a compound steel stanchion at either end, which in turn were supported on two compound girders 2 ft. 9 in. deep designed to spread the load in such a manner on the existing brick wall below as not to exceed the safe load per square foot. After the steel frame was inserted, steel folding wedges were inserted between the new compound girders and the existing 16 in. by 6-in. R.S.J., to make sure of taking all the load, and then the ends of the old steel joists were cut off and a stanchion erected on the new frame to receive the load coming from the girders at the head of the new opening on the floor above. After the insertion of the steelwork the new opening was safely cut and the whole operation carried out without any shoring inside the building whatever. The execution of the work, however, required some careful organisation, as the new steelwork itself weighed over 21 tons, and the whole of this had to be raised from the Embankment level, about 30 ft., and passed through small existing window openings.

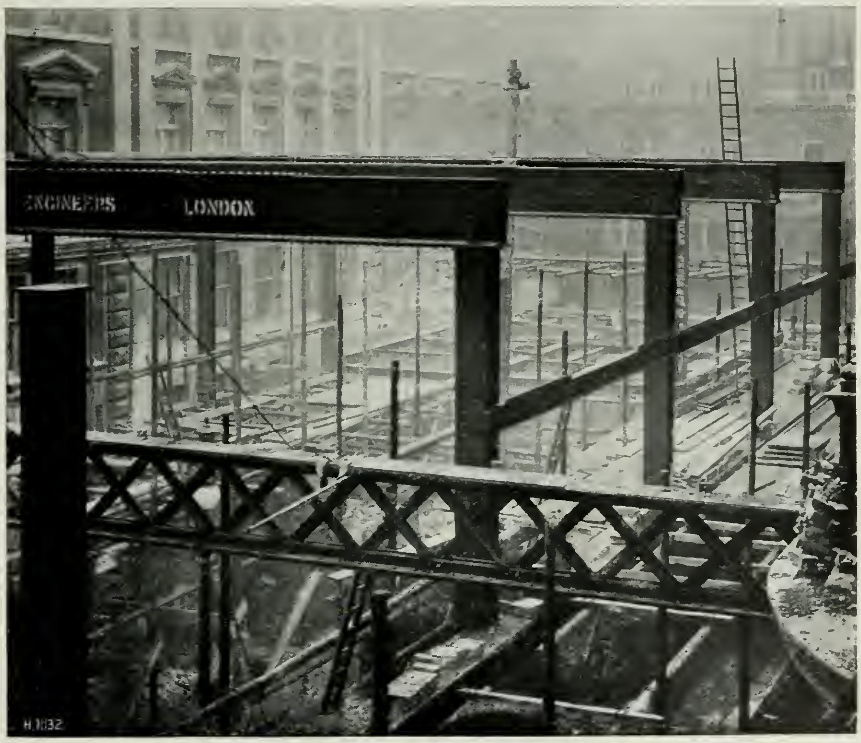


Fig. 8. General view of steelwork.
Additions to the Hotel Cecil.

The only space available for working at the Embankment level was an area about 8 ft. wide situated between the hotel and the Embankment gardens, and this being partially occupied by water tanks, etc., it was necessary to form a temporary timber roadway over them for the passage of the steelwork.

The contractors for the whole of this work are the Building Construction

Company, of Cockspur Street, and the steelwork was supplied by Messrs. Drew, Bear, Perks & Co. All the dormer windows, skylight aprons and curbs were formed of *in situ* concrete by the Siegwart Fireproof Floor Company, whose system of roofing was also adopted.

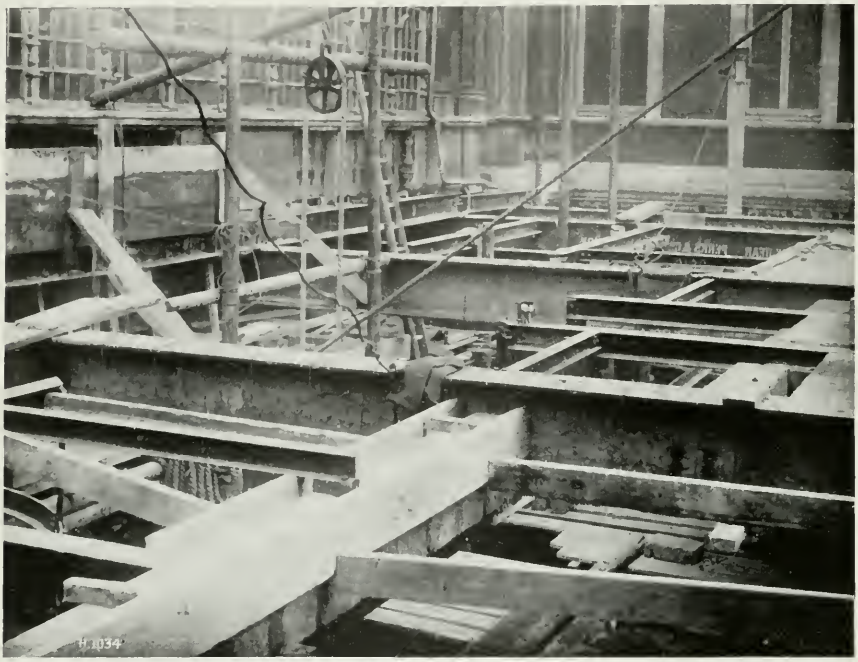


Fig. 9. Steel over Upper Palm Court.
ADDITIONS TO THE HOTEL CECIL.

ECONOMICAL DESIGN OF REINFORCED CONCRETE COLUMNS.

BASED ON THE SECOND REPORT OF THE ROYAL INSTITUTE OF BRITISH ARCHITECTS.

By HARRY JACKSON, A.M.Inst.C.E., A.R.S.M., etc.

We are constantly giving our readers articles on different methods of designing reinforced concrete work, and we think the present one dealing with the design of reinforced concrete columns should be useful to all students and others connected with this mode of construction.—ED.

WHEN a reinforced concrete column is subjected to a load, it is clear that both the concrete and the steel in the longitudinal bars are compressed, although the amount is very small. The assumption is made, on good grounds, that the amount of compression is the same in the concrete as in the steel.

The force required to compress the steel a given amount, as compared with the force required to compress the concrete an equal amount, depends upon their respective areas and upon the ratio of their *moduli of elasticity*. Taking this ratio of moduli of elasticity as 15, the resisting force in the steel for an equal area would be 15 times that of the concrete. Hence, if the working stress on the concrete is that shown in column 1 of Table I., the working stress in the steel will be that shown in column 2.

TABLE I.
SHOWING RELATION BETWEEN THE WORKING STRESS ON CONCRETE AND ON STEEL IN LONGITUDINAL BARS.

Working Stress on the Concrete in lb. per sq. in.	Corresponding Working Stress on Steel in lb. per sq. in.	Working Stress on the Concrete in lb. per sq. in.	Corresponding Working Stress on Steel in lb. per sq. in.
600	9,000	900	13,500
700	10,500	1,000	15,000
800	12,000	1,100	16,500
850	12,750		

The Committee of the Royal Institute of British Architects recommends that the steel should not be stressed to a greater amount than *15,000 lb.* per sq. in., and it will be seen, therefore, than unless the concrete can be stressed to *1,000 lb.* per sq. in., the steel will not be used as economically as it might, for with a given load the stress per sq. in. on the steel being lower, more steel would be required. For example:—

If a column were 9 in. square (measured over the longitudinal bars), and had four bars each $1\frac{1}{4}$ in. diameter, the concrete alone at 700 lb. per sq. in. would sustain $(9 \times 9) \times 700 - (4 \times 1{\cdot}227) \times 700 =$ ... 53,270 lb. = 23·8 tons.

The steel alone at 15×700 (10,500 lb.) would sustain
$4 \times 1{\cdot}227 \times 10{,}500 =$ ... ... 55,450 lb. = 24·7 tons.

108,720 lb. = 48·5 tons.

But suppose by some means the concrete is made to withstand a working stress of 1,000 lb., then the concrete would sustain

$(9 \times 9 - 4 \times 1{\cdot}227) \times 1{,}000 =$ ... ... 76,100 lb. = 34 tons.
The steel would sustain $4 \times 1{\cdot}227 \times 15{,}000 =$ ... 73,500 lb. = 32·8 tons.
149,600 lb. = 66·8 tons.

That is, for the same cross-section of column the load carried would be increased 19·3 tons, or 40 per cent.; and the same steel will, without any alteration in area or form, sustain 8·1 tons more than in the first case.

Put in another form, it may be considered thus: Suppose a column has to sustain a load of 66·8 tons. If the concrete can be so made or strengthened that it will sustain a crushing load of 1,000 lb. per square in., a cross-section of 11 in. square would suffice—that is, 9 in. over the longitudinal rods and 1 in. at each side of the rod extra.

If, however, the concrete can only sustain 700 lb. per square in., then with the same longitudinal rods a section of 13½ in. square would be required.

It is clear, therefore, that if by any means the working stress on the concrete can be increased to 1,000 lb., the working stress on the steel will be 15,000 lb., which is the maximum allowable, and considerable saving both in concrete and steel will ensue.

This saving in cost, however, is not nearly so important in many cases as saving of floor space, the columns for a five or six stories building becoming very large unless the fullest value of the concrete and the steel is made use of.

There are two ways in which the resistance of the concrete can be increased: (1) By increasing the richness of the mixture, thereby increasing its resistance to crushing; (2) by providing support to the concrete by means of lateral bindings.

To obtain the increased resistance by the first method requires an exceedingly rich mixture; in fact, with a mixture of one cement, one sand, and two shingle or broken stone, the stress allowable is only 900 lb. per square inch.

The second method, however, provides a ready method of increasing the compressive stress. Using the notation of the R.I.B.A. report, where

cp = the working compressive stress in plain concrete in lb. per square inch.
c = the safe compressive stress on the hooped core, in lb. per square inch.
f = a Form factor of 1, 0·75 or 0·5, depending upon whether the bindings are helical, circular hoops, or rectilinear.
s = a spacing factor or constant which varies with the pitch of the lateral bindings.
Vh = Volume of hooped reinforcement in cubic inches.
V = Volume of hooped core in cubic inches measured for the same length of column as Vh.
$r = \frac{Vh}{V}$ = the ratio of volumes, *i.e.*, the ratio of the volume of the helical or horizontal reinforcement to the volume of the hooped core.

$$\text{Then } c = cp\,(1 + fsr) \quad \ldots \quad \ldots \quad \text{Equation 1.}$$

Of these, cp is known for any particular mixing of concrete; f is given above; r is obtained from the volumes Vh and V, which depend upon the cross-section of the binding wires, the cross-section of the column, and the pitch of the lateral bindings. The value s is obtained from the equation

$$s = 48 - 80\frac{p}{d} \quad \ldots \quad \ldots \quad \text{Equation 2.}$$

So that with any particular diameter of round column or side of square column and a particular pitch of binding wire, it is necessary to calculate *s* from equation 2 before *c* can be obtained from equation 1.

This has been done in Table II. for columns of diameters from 3 in. to 12 in., and for pitches from ½ to 8 in.

TABLE II.

HOOPED COLUMNS.

Value of $s=48-80\frac{p}{d}$. Value of p=(the pitch).

Diameter of Column.	½ in.	1 in.	1¼ in.	1½ in.	1¾ in.	2 in.	2½ in.	3 in.	4 in.	5 in.	6 in.	7 in.	8 in.
3 in.	35	21	15	8	1	0	—	—	—	—	—	—	—
4 in.	38	28	23	18	13	8	0	—	—	—	—	—	—
5 in.	40	32	28	24	20	16	8	0	—	—	—	—	—
6 in.	41	35	31	28	25	21	15	8	0	—	—	—	—
7 in.	42	36	34	31	28	25	19	14	2	—	—	—	—
8 in.	43	38	35	32	30	28	23	18	8	0	—	—	—
9 in.	43	39	37	35	32	30	26	21	12	3	—	—	—
10 in.	44	40	38	36	34	32	28	24	16	8	0	—	—
11 in.	44	41	39	37	35	33	30	26	19	12	4	0	—
12 in.	44	41	40	38	36	35	31	28	21	15	8	1	0

It is laid down by the R.I.B.A. that *s* shall not be taken as more than 32, although the reason is not very clear, and consequently we see at once from the Table II. that the minimum pitches allowable for particular sizes of hooped cores are as in Table III.

TABLE III.

MINIMUM PITCH OR DISTANCE APART OF LATERALS FOR PARTICULAR DIAMETERS OF COLUMNS (HOOPED CORE, MEASURED TO THE OUTSIDE OF LONGITUDINAL RODS).

Diameter of Column.	Minimum Pitch allowable.	Diameter of Column.	Minimum Pitch allowable.
3 in.	¾ in.	8 in.	1½ in.
4 in.	¾ in.	9 in.	1¾ in.
5 in.	1 in.	10 in.	2 in.
6 in.	1¼ in.	11 in.	2 in.
7 in.	1½ in.	12 in.	2½ in.

Having obtained *s*, it is next necessary to find the value of *r*. This depends not only upon the pitch of the lateral bindings, but also on the cross-section of the same. These particular sizes are taken for the following tables, namely :—

Imperial Wire Gauge (I.W.G.) No. 10 ; $\frac{1}{8}$ in. dia. = ·128 in. area (*a*) = ·01287 sq. in.
" " " No. 6 ; $\frac{3}{16}$ in. dia. = ·192 in. " = ·029 sq. in.
" " " No. 3 ; $\frac{1}{4}$ in. dia. = ·250 in. " = ·050 sq. in.

and the particulars for each of these are indicated by the following letters :—

I.W.G. No. 10 ... ... A
I.W.G. No. 6 ... ... B
I.W.G. No. 3 C

Whether the column is round or square—

$$r=\frac{Vh}{V}$$

$$=\frac{4a}{dp}$$

For in the case of a round column, taking a length of column equal to p (the distance between two consecutive laterals),

$$r = \frac{Vh}{V} = \frac{ad\pi}{\frac{d^2\pi}{4} \times p} = \frac{4a}{dp}$$

For square columns—

$$r = \frac{Vh}{V} = \frac{a \times 4 \times d}{d^2 p} = \frac{4a}{dp}$$

The values of r for the particular columns, pitches, and areas of binding wires taken are given in Table IV.

TABLE IV.

HOOPED ROUND OR SQUARE COLUMNS.

Value of $r = \frac{4a}{dp}$

Value of $a =$ I.W.G. 10, dia. $\frac{1}{8}$-in. = ·128 in.; a = ·01287 (A).
I.W.G. 6, dia. $\frac{3}{16}$-in. = ·192 in.; a = ·029 (B).
I.W.G. 3, dia. $\frac{1}{4}$-in. = ·25 in.; a = ·050 (C).

Values of p = Pitch of Lateral Binding Wire.

Diameter of Column.	Particulars of Laterals.	½ in.	1 in.	1¼ in.	1½ in.	1¾ in.	2 in.	2½ in.	3 in.	4 in.	5 in.	6 in.	7 in.
3 in.	A	·0343	·0172	·0137	·0114	·0098	—	—	—	—	—	—	—
	B	·0773	·0386	·0310	·0258	·0221	—	—	—	—	—	—	—
	C	·1334	·0668	·0534	·0445	·0381	—	—	—	—	—	—	—
4 in.	A	·0257	·0129	·0103	·0086	·0074	·0064	—	—	—	—	—	—
	B	·058	·0290	·0232	·0193	·0166	·0145	—	—	—	—	—	—
	C	·100	·0500	·0400	·0334	·0286	·0250	—	—	—	—	—	—
5 in.	A	—	·0103	·0082	·00687	·0059	·0052	·00412	—	—	—	—	—
	B	—	·0232	·0186	·0155	·0132	·0116	·0093	—	—	—	—	—
	C	—	·0400	·0320	·0267	·0229	·0200	·0160	—	—	—	—	—
6 in.	A	—	—	·0068	·0057	·0049	·0043	·0035	·00286	—	—	—	—
	B	—	—	·0155	·0129	·0110	·0097	·0077	·00645	—	—	—	—
	C	—	—	·0267	·0228	·0190	·0167	·0133	·0111	—	—	—	—
7 in.	A	—	—	—	·00491	·0042	·00368	·00294	·00245	·00184	—	—	—
	B	—	—	—	·0111	·0095	·0083	·0066	·0055	·00415	—	—	—
	C	—	—	—	·0191	·0163	·0143	·0114	·0095	·00715	—	—	—
8 in.	A	—	—	—	·00429	·00368	·00322	·00257	·00215	·00161	—	—	—
	B	—	—	—	·00967	·00830	·00726	·00581	·00483	·00363	—	—	—
	C	—	—	—	·0167	·0143	·0125	·01000	·00832	·00626	—	—	—
9 in.	A	—	—	—	·00382	·00327	·00286	·00229	·00191	·00143	·00114	—	—
	B	—	—	—	·00860	·00737	·00644	·00516	·00430	·00323	·00258	—	—
	C	—	—	—	·0148	·0127	·0110	·0089	·00741	·00556	·00445	—	—
10 in.	A	—	—	—	—	—	·00257	·00206	·00172	·00129	·00103	—	—
	B	—	—	—	—	—	·00581	·00464	·00387	·00290	·00232	—	—
	C	—	—	—	—	—	·01000	·00800	·00667	·00500	·00400	—	—
11 in.	A	—	—	—	—	—	·00234	·00187	·00156	·00117	·00094	·00078	—
	B	—	—	—	—	—	·00528	·00422	·00352	·00264	·00211	·00176	—
	C	—	—	—	—	—	·00910	·00727	·00606	·00454	·00364	·00303	—
12 in.	A	—	—	—	—	—	—	·00173	·00143	·00107	·00086	·00072	·00061
	B	—	—	—	—	—	—	·00387	·00322	·00242	·00193	·00161	·00138
	C	—	—	—	—	—	—	·00667	·00556	·00417	·00333	·00278	·00238

NOTE.—The rows of figures opposite A are for laterals of I.W.G. No. 10; B are for laterals of I.W.G. No. 6; C are for laterals of I.W.G. No. 3.

It will be seen that values of r have not been inserted for pitches less than the minimum shown in Table III. for particular columns, as the R.I.B.A. rules prohibit the use of these pitches with the columns in question.

We have now obtained all the values required by Equation 1 to enable c to be found for the particular columns taken.

Inserting these values, we get in Table V. the values of $[1 + fsr]$ for *round columns*, and in Table VI. for *square columns*.

ROUND COLUMNS WITH SPIRAL BINDINGS.

TABLE V.

CIRCULAR HOOPED COLUMNS (SPIRALLY WOUND).

Value of $[1+fsr]$.
$f=1$

Laterals
I.W.G. No. 10, ⅛ in. = ·128 in. dia.; $a=·01287$ (A).
I.W.G. No. 6, $\frac{3}{16}$ in. = ·192 in. dia.; $a=·029$ (B).
I.W.G. No. 3, ¼ in. = ·25 in. dia.; $a=·050$ (C).

Values of $[1+fsr]$ for laterals having pitch of ½ in. to 7 in.

Diameter of Column.	Particulars of Laterals.	½ in.	1 in.	1¼ in.	1½ in.	1¾ in.	2 in.	2½ in.	3 in.	4 in.	5 in.	6 in.	7 in.
3 in.	A	1·20	1·361	1·205	1·091	1·010	—	—	—	—	—	—	—
	B	3·705	1·811	1·465	1·206	1·022	—	—	—	—	—	—	—
	C	4·969	2·402	1·800	1·356	1·038	—	—	—	—	—	—	—
4 in.	A	1·977	1·361	1·237	1·155	1·096	1·051	—	—	—	—	—	—
	B	1·204	1·812	1·534	1·348	1·215	1·116	—	—	—	—	—	—
	C	4·800	2·400	1·920	1·601	1·371	1·200	—	—	—	—	—	—
5 in.	A	—	1·340	1·230	1·165	1·118	1·083	1·033	—	—	—	—	—
	B	—	1·742	1·521	1·372	1·264	1·186	1·075	—	—	—	—	—
	C	—	2·280	1·896	1·641	1·458	1·320	1·128	—	—	—	—	—
6 in.	A	—	—	1·211	1·159	1·122	1·090	1·053	1·023	—	—	—	—
	B	—	—	1·480	1·361	1·275	1·204	1·116	1·052	—	—	—	—
	C	—	—	1·827	1·624	1·475	1·357	1·200	1·089	—	—	—	—
7 in.	A	—	—	—	1·152	1·118	1·092	1·056	1·034	1·004	—	—	—
	B	—	—	—	1·344	1·266	1·207	1·125	1·077	1·008	—	—	—
	C	—	—	—	1·591	1·456	1·357	1·217	1·133	1·014	—	—	—
8 in.	A	—	—	—	1·137	1·110	1·091	1·059	1·039	1·013	—	—	—
	B	—	—	—	1·309	1·249	1·203	1·134	1·087	1·029	—	—	—
	C	—	—	—	1·534	1·429	1·350	1·230	1·150	1·051	—	—	—
9 in.	A	—	—	—	—	1·105	1·086	1·059	1·040	1·017	1·003	—	—
	B	—	—	—	—	1·236	1·193	1·134	1·090	1·039	1·008	—	—
	C	—	—	—	—	1·406	1·330	1·231	1·156	1·067	1·014	—	—
10 in.	A	—	—	—	—	—	1·082	1·058	1·041	1·021	1·008	—	—
	B	—	—	—	—	—	1·186	1·130	1·093	1·046	1·019	—	—
	C	—	—	—	—	—	1·320	1·224	1·160	1·080	1·032	—	—
11 in.	A	—	—	—	—	—	1·077	1·056	1·040	1·022	1·011	1·003	—
	B	—	—	—	—	—	1·174	1·127	1·091	1·050	1·025	1·007	—
	C	—	—	—	—	—	1·300	1·218	1·158	1·086	1·044	1·012	—
12 in.	A	—	—	—	—	—	—	1·055	1·040	1·022	1·013	1·006	1·001
	B	—	—	—	—	—	—	1·119	1·090	1·051	1·029	1·014	1·002
	C	—	—	—	—	—	—	1·206	1·156	1·084	1·050	1·024	1·003

PARTICULARS DEDUCED FROM TABLE V.

Columns.	Working Stress on Concrete.	Lateral Bindings.
3 in., 4 in., 5 in., and 6 in.	1,000 lb./sq. in.	Laterals: No. 1, I.W.G. No. 6, 1¼-in. pitch.
6 in., 7 in., 8 in., and 9 in.	1,000 lb./sq. in.	Laterals: No. 1, I.W.G. No. 3, or No. 2, I.W.G. No. 6 (dia. ·192), 1¾-in. pitch.
9 in., 10 in., and 11 in.	900 lb./sq. in.	Laterals: No. 1, I.W.G. No. 3, or No. 2, I.W.G. No. 6 (dia. ·192), 2-in. pitch.
12 in.	850 lb./sq. in.	Laterals: No. 1, I.W.G. No. 3, or No. 2, I.W.G. No. 6 (dia. ·192), 2½-in. pitch.

Plain concrete = 700 lb./sq. in.

Dealing first with the round columns, take the columns having 3 in., 4 in., 5 in., and 6 in. diameter, it will be seen that this condition $c=1{,}000$ lb. sq. in. is fulfilled when I.W.G. No. 6 binding is used at 1¼ in. pitch, for then the value of $[1+fsr]$ most nearly approximates to $\frac{1000}{700}=1·43$.

Similarly in columns 6 in., 7 in., 8 in., and 9 in. diameter, respectively, the same result is obtained with I.W.G. No. 3, 1¾ in. pitch. As, however I.W.G. No. 3 is rather difficult to bend, two I.W.G. No. 6 wires close together may be used.

When we get to 10 in. and 11 in. diameter it will be seen that there is no figures more than $\frac{1000}{700} = 1{\cdot}43$, and it is not possible, therefore, with columns of these dimensions to get a working stress of 1,000 lb. per sq. in. when complying with the R.I.B.A. conditions.

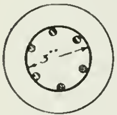
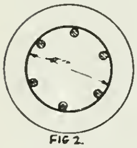
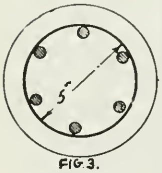
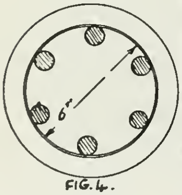
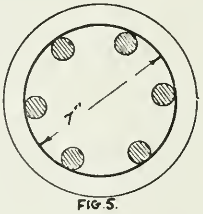
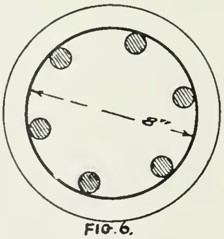
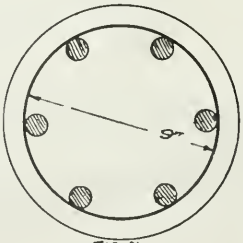
If 900 lb. per sq. in. on the concrete is used, the value of $[1 + fsr] = \frac{900}{700} = 1{\cdot}29$ is obtained by using I.W.G. No. 3, 2 in. pitch, and it will be seen that this can be used with advantage in a 9-in. column also.

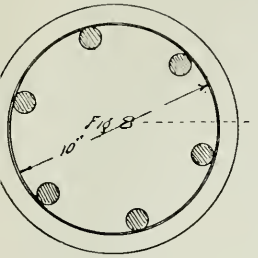
For the 12-in. diameter column it will be seen that no figure of 1·29 can be obtained, and consequently no stress of 900 lb. on the concrete can be used. If, however, the value for $[1 + fsr]$ of 1·206 be taken the stress on the concrete would be 850 lb.

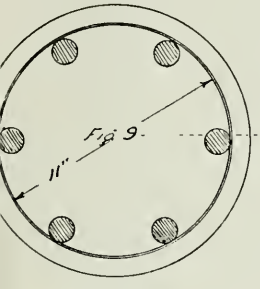
It will be noticed by inspection of the figures for pitches of 3 in., 4 in., 5 in., 6 in., and 7 in. that no material increase in concrete stress is obtained, and the lower values of stresses on the concrete and the steel of 700 lb. and 10,500 lb. respectively would be used.

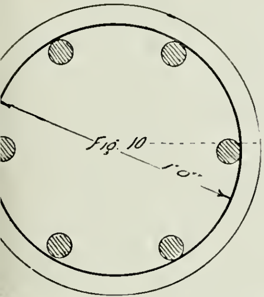
Having now obtained the possible lateral binding and consequent concrete and steel stress for particular column diameters, we can proceed to determine the loads the columns would withstand. These are shown in *Figs.* 1 to 10.

If columns are required to sustain heavier loads, then three or more smaller diameter columns can be built up so as to take the particular load required, as in *Figs.* 11 and 12.

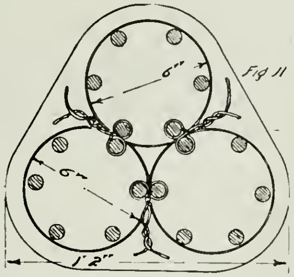
	Longitudinal Bars.		Helical Bindings.				Total Load in Tons.
	No.	Dia. In.	No.	I.W.G.	Dia. In.	Pitch In.	
 FIG. 1.	6	$\frac{3}{8}$	1	6	$\frac{3}{16}$	$1\frac{1}{4}$	$7\frac{1}{2}$
 FIG. 2.	6	$\frac{3}{8}$	1	6	$\frac{3}{16}$	$1\frac{1}{4}$	10
 FIG. 3.	6	$\frac{1}{2}$	1	6	$\frac{3}{16}$	$1\frac{1}{4}$	$16\frac{1}{2}$
 FIG. 4.	6	$\frac{3}{4}$	1	6	$\frac{3}{16}$	$1\frac{1}{4}$	30
 FIG. 5.	6	1	2	6	$\frac{3}{16}$	$1\frac{3}{4}$	49
 FIG. 6.	6	1	2	6	$\frac{3}{16}$	$1\frac{3}{4}$	54
 FIG. 7.	6	1	2	6	$\frac{3}{16}$	$1\frac{3}{4}$	60

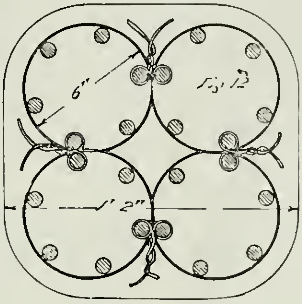






Longitudinal Bars.		Helical Bindings.				Total Load in Tons.
No	Dia. In.	No.	I.W.G.	Dia. In.	Pitch In.	
6	1	2	6	$\frac{3}{16}$	2	60
6	$1\frac{1}{8}$	2	6	$\frac{3}{16}$	2	74
6	$1\frac{1}{8}$	2	6	$\frac{3}{16}$	$2\frac{1}{2}$	77





Longitudinal Bars.		Helical Bindings.			
No.	Dia. In.	No.	I.W.G.	Dia. In.	Pitc In.
18	$\frac{3}{4}$	1	6	$\frac{3}{16}$	$1\frac{1}{4}$
24	$\frac{3}{4}$	1	6	$\frac{3}{16}$	$1\frac{1}{4}$

SQUARE COLUMNS WITH RECTILINEAR LATERALS.

Turning now to square columns with rectilinear laterals, the value of r in the equation 1, $c=cp\,[1+fsr]$, for any given distance over the longitudinal reinforcement and pitch of winding, is the same as for a circular column of the same diameter, and is given in Table IV.

The value of s is the same as for circular columns, and is given in Table II. The value of f is ·5 instead of 1·0 for circular columns with helical winding. Putting these figures in the expression $[1+fsr]$ we obtain Table VI. :—

TABLE VI

Square Columns with Rectilinear Laterals.

Value of $[1+fsr]$
$f=0{\cdot}5$

Laterals
I.W.G. No. 10—$\frac{1}{8}$ in. dia. ; $a={\cdot}01287$ (A).
I.W.G. No. 6—$\frac{3}{16}$ in. dia. ; $a={\cdot}029$ (B).
I.W.G. No. 3—$\frac{1}{4}$ in. dia. ; $a={\cdot}050$ (C).

Side of Square.	Particulars of Laterals.	½ in.	1 in.	1¼ in.	1½ in.	1¾ in.	2 in.	2½ in.	3 in.	4 in.	5 in.	6 in.	7 in.
3 in.	A	1·600	1·180	1·102	1·046	1·005	—	—	—	—	—	—	—
	B	2·353	1·405	1·233	1·103	1·011	—	—	—	—	—	—	—
	C	3·335	1·701	1·400	1·178	1·019	—	—	—	—	—	—	—
4 in.	A	1·608	1·180	1·119	1·078	1·044	1·026	—	—	—	—	—	—
	B	2·102	1·406	1·267	1·174	1·107	1·058	—	—	—	—	—	—
	C	2·900	1·700	1·460	1·301	1·186	1·100	—	—	—	—	—	—

TABLE VI.—*Continued*

Side of Square	Particulars of Laterals.	½ in.	1 in.	1¼ in.	1½ in.	1¾ in.	2 in.	2½ in.	3 in.	4 in.	5 in.	6 in.	7 in.
5 in.	A	—	1·170	1·115	1·083	1·059	1·042	1·017	—	—	—	—	—
	B	—	1·371	1·261	1·186	1·132	1·093	1·038	—	—	—	—	—
	C	—	1·640	1·448	1·321	1·229	1·160	1·064	—	—	—	—	—
6 in.	A	—	—	1·106	1·080	1·061	1·045	1·027	1·012	—	—	—	—
	B	—	—	1·240	1·181	1·138	1·102	1·058	1·026	—	—	—	—
	C	—	—	1·414	1·312	1·238	1·179	1·100	1·045	—	—	—	—
7 in.	A	—	—	—	1·076	1·059	1·046	1·028	1·017	1·002	—	—	—
	B	—	—	—	1·172	1·133	1·104	1·063	1·038	1·004	—	—	—
	C	—	—	—	1·296	1·228	1·179	1·109	1·067	1·007	—	—	—
8 in.	A	—	—	—	1·069	1·053	1·046	1·030	1·020	1·007	—	—	—
	B	—	—	—	1·155	1·125	1·102	1·067	1·044	1·015	—	—	—
	C	—	—	—	1·267	1·215	1·175	1·115	1·075	1·026	—	—	—
9 in.	A	—	—	—	—	1·052	1·043	1·030	1·020	1·008	1·002	—	—
	B	—	—	—	—	1·118	1·097	1·062	1·045	1·020	1·004	—	—
	C	—	—	—	—	1·203	1·165	1·116	1·078	1·034	1·007	—	—
10 in.	A	—	—	—	—	—	1·041	1·029	1·020	1·011	1·004	—	—
	B	—	—	—	—	—	1·093	1·065	1·047	1·023	1·010	—	—
	C	—	—	—	—	—	1·160	1·112	1·080	1·040	1·016	—	—
11 in.	A	—	—	—	—	—	1·045	1·028	1·020	1·011	1·006	1·002	—
	B	—	—	—	—	—	1·087	1·064	1·046	1·025	1·013	1·004	—
	C	—	—	—	—	—	1·150	1·109	1·079	1·043	1·022	1·006	—
12 in.	A	—	—	—	—	—	—	1·027	1·020	1·011	1·007	1·003	1·001
	B	—	—	—	—	—	—	1·060	1·045	1·026	1·015	1·007	1·001
	C	—	—	—	—	—	—	1·103	1·078	1·042	1·025	1·012	1·002

PARTICULARS DEDUCED FROM TABLE VI.

Side of Square.	Working Stress on Concrete.	Lateral Binding Wires.
3 in., 4 in. and 5 in.	950 lb.	I.W.G., No. 6, 1 in. pitch.
6 in. and 7 in.	900 lb.	I.W.G., No. 3, 1½ in. pitch.
8 in. and 9 in.	850 lb.	I.W.G., No. 3, 1¾ in. pitch.
10 in. and 11 in.	800 lb.	I.W.G., No. 3, 2 in. pitch.
12 in.	750 lb.	I.W.G., No. 3, 3 in. pitch.

Plain concrete 700 lb./sq. in.

An inspection of the table shows that for 3-in. and 4-in. square columns the value of $[1 + fsr]$ is more than $\frac{1000}{700} = 1{\cdot}43$, when ⅛-in. laterals ½ in. apart are used.

The use of any laterals less than $\frac{3}{16}$ in. diameter is not approved by the rules, however; and with $\frac{3}{16}$-in. laterals it is not possible to get the 1,000 lb. per sq. in. compressive stress on the concrete, except for columns 3 in. and 4 in. square.

For 3-in., 4-in. and 5-in. square columns the most suitable stress is seen to be 950 lb., with laterals of I.W.G. No. 6, 1 in. apart.

For 6-in. and 7-in. columns stress on concrete is 900 lb. with I.W.G. No. 3, 1½ in. apart.

With 8-in. and 9-in. columns stress on concrete is 850 lb., I.W.G. No. 3, 1¾ in. apart.

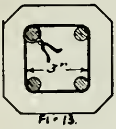
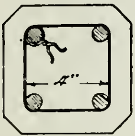
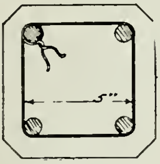
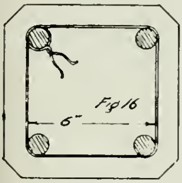
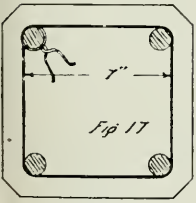
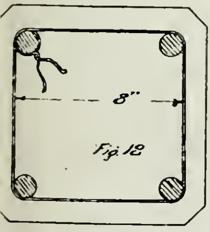
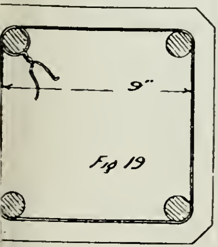
With 10-in. and 11-in. columns stress on concrete is 800 lb., I.W.G. No. 3, 2 in. apart.

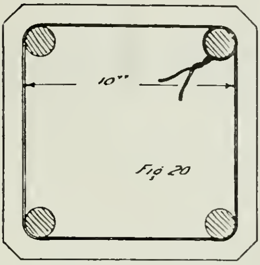
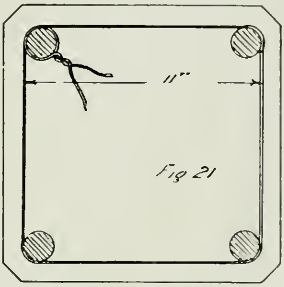
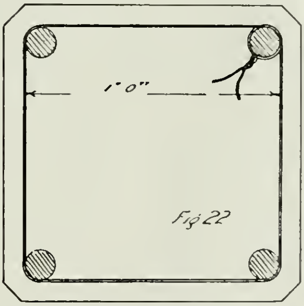
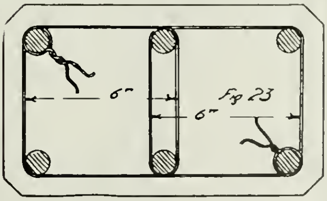
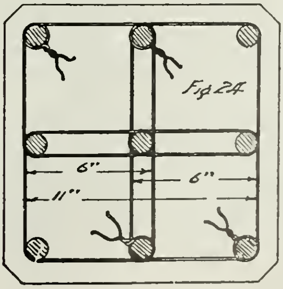
With 12-in. columns stress on concrete is 750 lb., I.W.G. No. 3, 3 in. apart.

Again, as in the circular columns, no increase in the concrete stress is obtained by using pitches greater than 3 in.

Figures 13-22 show sections of square columns having the steel economically used, with the corresponding loads they will carry indicated as with circular columns :—

MINIMUM CROSS-SECTION SQUARE COLUMNS WITH RECTILINEAR LATERALS.

	Longitudinal Bars.		Rectilinear Laterals.				Total Load in Tons
	No.	Dia. In.	No.	I.W.G.	Dia. In.	Pitch. In.	
 Fi • 13.	4	½	1	6	3/16	1	8¾
 FIG. 14.	4	¾	1	6	3/16	1	18
 FIG. 15.	4	⅞	1	6	3/16	1	26
	4	1	1	3	¼	1½	33¼
	4	1⅛	1	3	¼	1½	43½
	4	1⅛	1	3	¼	1¾	47
	4	1¼	1	3	¼	1¾	59

	Longitudinal Bars.		Rectilinear Laterals.				Total Load
	No.	Dia. In.	No.	I.W.G.	Dia. In.	Pitch. In.	
	4	1⅜	1	3	¼	2	
	4	1⅜	1	3	¼	2	7
	4	1⅜	1	3	¼	3	7
	6	1	1	3	¼	1½	5
	9	1	1	3	¼	1½	8

It is possible to combine two or more of the smaller cross-sections and so obtain any particular load in excess of 78 tons, that being the load on the 12-in. square columns, with No. 4 1⅜ in. bars and laterals of I.W.G. No. 3 (¼ in. diameter), spaced 3 in. apart; and two cases of combination are shown in *Figs.* 23 and 24.

TABLE VII. See figures 1 to 10.

MINIMUM CROSS-SECTION COLUMNS WITH HELICAL LATERALS.

Diameter of Columns over Binding Wires.	Particulars of Laterals. Size and No. of Wires.				Area inside Laterals.	Stress per sq. in. on Concrete.	No. and Size of Longitudinals.			Load on Concrete.	Load on Longitudinals.	Total Load.
	No.	Dia.	I.W.G.	Pitch.			No.	Dia.	Total Area.			
in.				in.	sq. in.	lb.		in.	sq. in.	Tons.	Tons.	Tons.
3	1	·192	No. 6	1¼	7·07	1,000	6	⅜	·66	3·14	4·43	7·57
4	1	·192	No. 6	1¼	12·57	1,000	6	⅜	·66	5·61	4·43	10·04
5	1	·192	No. 6	1¼	19·64	1,000	6	½	1·18	8·79	7·90	16·69
6	1 2	·192	No. 6	1¼ 1¾	28·27	1,000	6	¾	2·65	12·62	17·70	30·32
7	2	·192	No. 6	1¾	38·49	1,000	6	1	4·72	17·20	31·60	48·80
8	2	·192	No. 6	1¾	50·26	1,000	6	1	4·72	22·40	31·60	54·00
9	2 2	·192	No. 6	1¾ 2	63·62	1,000 900	6	1 1	4·72 4·72	28·50 25·60	31·60 28·40	60·10 54·00
10	2	·192	No. 6	2	78·54	900	6	1	4·72	31·60	28·40	60·00
11	2	·192	No. 6	2	95·03	900	6	1⅛	5·96	38·20	36·00	74·20
12	2	·192	No. 6	2½	113·10	850	6	1⅛	5·96	42·90	33·95	76·85

TABLE VIIA. See figures 13 to 22.

MINIMUM AREA CROSS SECTIONS OF SQUARE COLUMNS WITH RECTILINEAR LATERALS.

Diameter over Rods.	Particulars of Laterals.				Area of Concrete inside Laterals.	Stress per sq. in. allowed on Concrete.	No. and Size of Longitudinal Bars.			Load on Concrete.	Load on Longitudinals.	Total Load.
	No.	Dia.	I.W.G.	Pitch.			No.	Dia.	Total Area.			
in.		in.	No.	in.	sq. in.	lb.		in.	sq. in.	Tons.	Tons.	Tons
3	1	3/16	6	1	9	950	4	½	·78	3·82	4·97	8·79.
4	1	3/16	6	1	16	950	4	¾	1·76	6·79	11·19	17·98
5	1	3/16	6	1	25	950	4	⅞	2·40	10·60	15·25	25·85
6	1	¼	3	1½	36	900	4	1	3·14	14·45	18·92	33·37
7	1	¼	3	1½	49	900	4	1⅛	3·97	19·70	23·96	43·66
8	1	¼	3	1¾	64	850	4	1⅛	3·97	24·30	22·59	46·89
9	1	¼	3	1¾	81	850	4	1¼	4·91	30·70	27·98	58·68
10	1	¼	3	2	100	800	4	1⅜	5·94	35·70	31·80	67·50
11	1	¼	3	2	121	800	4	1⅜	5·94	43·20	31·80	75·00
12	1	¼	3	3	144	750	4	1⅜	5·94	48·20	29·80	78·00

Plain Concrete 700 lb./sq. in.

It may be desirable to vary from the particular columns shown in the figures, and for the purpose of seeing whether it is more desirable to alter the concrete or the steel, Tables VII. and VIIA. are given.

COMPARISON OF COSTS BETWEEN ROUND AND SQUARE COLUMNS.

Having now determined the most economical sections with respect to the amount of material, it is desirable to compare the costs of the particular columns for the same load-carrying capacity.

TABLE VIII.

COMPARATIVE COST OF SQUARE AND ROUND COLUMNS.

Length taken at 1 ft.

Diameter Overall, including 1-in. Cover of Rods.		Total Volume of Concrete.		Total Volume of Steel in Square Columns.			Volume of Steel in Round Columns.			Concrete per ft. per ton of Load.		Steel per ft. per ton of Load.		Cost per ton per ft. Long.					
														Square Columns.			Round Column.		
Over all.	Over Rods.	Square.	Round.	Longitudinal.	Lateral.	Total.	Longitudinal.	Lateral.	Total.	Square.	Round.	Square.	Round.	Concrete.	Steel.	Total.	Concrete.	Steel.	Total.
in.	in.	cu. ft.	cu. ft.	cu. in.	cu. in.	cu. in.	cu. in.	cu. in.	cu. in.	cu. ft.	cu. ft.	cu. in.	cu. in.	d.	d.	d.	d.	d.	d.
5	3	·174	·136	9·36	4·18	13·54	7·92	2·66	10·58	·0198	·0180	1·54	1·40	·495	·644	1·139	·450	·586	1·036
6	4	·250	·196	21·12	5·57	26·69	7·92	3·50	11·42	·0139	·0195	1·48	1·14	·347	·620	·967	·487	·477	·964
7	5	·341	·267	28·80	6·96	35·76	14·18	4·37	18·55	·0132	·0160	1·44	1·11	·330	·602	·932	·400	·464	·864
8	6	·444	·349	37·70	9·60	47·30	31·80	3·75	35·55	·0132	·0115	1·43	1·17	·330	·597	·927	·288	·490	·778
9	7	·562	·442	47·60	11·20	58·80	56·70	4·37	61·07	·0129	·0091	1·35	1·26	·323	·565	·888	·228	·527	·755
10	8	·694	·546	47·60	10·96	58·56	56·70	4·99	61·69	·0148	·0101	1·25	1·14	·370	·524	·894	·252	·477	·729
11	9	·841	·662	58·90	12·32	71·22	56·70	4·91	61·61	·0143	·0110	1·21	1·03	·358	·506	·864	·275	·431	·706
12	10	1·000	·785	71·20	12·00	83·20	56·70	5·46	62·16	·0148	·0131	1·23	1·03	·370	·514	·884	·327	·431	·758
13	11	1·173	·921	71·20	13·20	84·40	71·50	6·01	77·51	·0157	·0124	1·13	1·04	·392	·473	·865	·310	·435	·745
14	12	1·361	1·068	71·20	9·60	80·80	71·50	5·25	76·75	·0175	·0139	1·03	1·00	·437	·431	·868	·348	·418	·766
								Average	.. ..	·0150	·0135	1·309	1·132	·375	·547	·922	·337	·473	·811

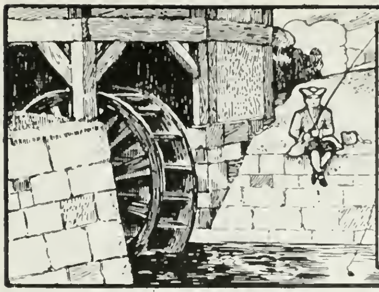
NOTE.—Prices for Steel (fixed), £13 15s. per ton = ·418d. per cu. in.; Prices for Concrete = 2s. 1d. per cu. ft.

To do this it has been assumed that the price of the steel in position is £13 15s. per ton, and of the concrete 2s. 1d. per cubic foot. The actual cost of each column per foot of length, divided by the load it will carry, gives the cost per foot of length for every ton carried, which forms a satisfactory basis for comparison.

Table VIII. shows the actual amount of steel and concrete used in the particular columns as set out, and the cost per ton of load supported per foot length of column.

It will be seen that in practically all cases the round column is cheaper than the square column, the price varying from ·706d. per ton per linear foot of column for a 9-in. column, to 1·036d. for a 3-in. column.

From this it will be seen that, if a large column is to be built, requiring to sustain, say, 240 tons, the most economical arrangement will be that of four units of 9 in. diameter each, consisting of six 1-in. diameter rods with helical winding laterals, No. 2 (close together), I.W.G. No. 6, $\frac{3}{16}$ in. diameter, $1\frac{3}{4}$ in. pitch.

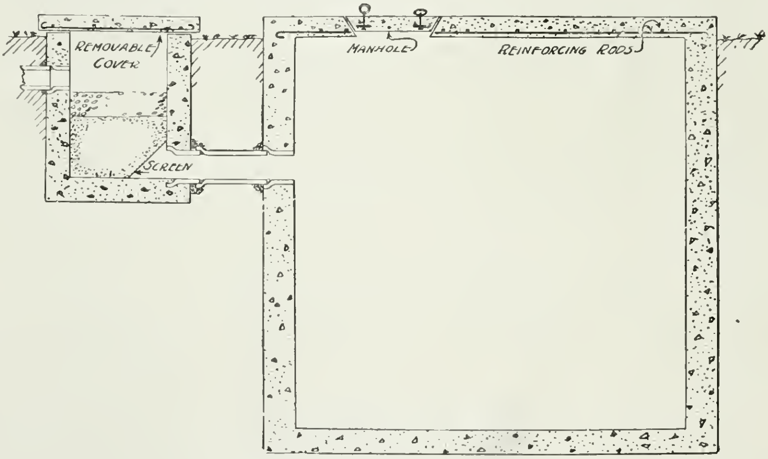


UNDERGROUND CISTERNS OF CONCRETE.

The following information as to the method of making cheap and effective underground cisterns of concrete has been issued by the Association of American Portland Cement Manufacturers, and should be found very useful by estate and farm owners.—ED.

Underground cisterns may be built in any shape desired. For the same amount of concrete, round cisterns (with a depth equal to the diameter) will hold the most water. However, since round forms are not easy to frame, most cisterns are made square. To illustrate the usual method of building, consider the construction of a cistern 8 ft. by 8 ft. by 8 ft. in the clear, with walls 8 in. thick and with a capacity of 121 barrels. Before beginning the work have all the materials on hand, slightly more than required.

Locate the cistern in the most convenient place. Since the walls are 8 in. thick, lay out the hole 9 ft. 4 in. square. The concrete bottom is 6 in. thick. Therefore dig the pit to the depth of 8 ft. 6 in. Since the concrete cover or platform will be 5 in. thick, the top of the cistern will be 5 in. above the ground, which is a desirable feature.

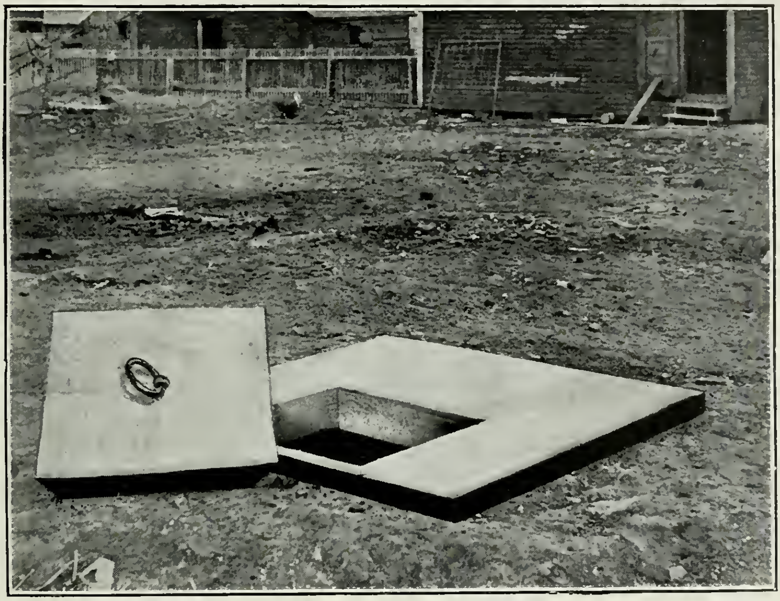


Cross Section of Underground Cistern and Filter.

If the earth walls stand firm, only an inside form will be needed; otherwise make a similar form for the outside of the walls. This form should be built (each side separately) previous to digging the pit, so that it may be quickly erected and the cistern finished before a possible shower makes the hole muddy.

For siding, use 1-in. boards on 2-in. by 4-in. uprights spaced 2 ft. As soon as the hole is dug, set up the forms on 6-in. cubes or bricks of concrete, with 1-in. removable wedges between the forms and the bricks. The side wall forms will later support the platform on which the concrete cover will be built; and unless these wedges are used the forms will bind and be difficult to remove. Join the forms at the corners so that they can be easily taken down after the cistern is finished. Against the earthen walls, and braced to the inside forms, set 1-in. boards, 12 in. wide, so as to prevent dirt from crumbling and falling into the cistern.

Mix the concrete 1 part Portland cement to 2 parts sand to 4 parts crushed rock. In measuring the quantities, consider 1 bag of cement equal to 1 cubic foot. If gravel is to be used, proportion the concrete 1 part cement to 4 parts clean gravel. For the floor have the concrete just wet enough to flush a little cement mortar to the surface when tamped into place. See that there is an abundance of mortar around the concrete blocks supporting the forms. Immediately begin filling the sidewall forms with concrete mixed mushy wet and placed in 8-in. to 10-in. layers around the entire cistern. Be careful not to shovel the concrete against the earthen sides, as dirt in concrete is liable to make a leaky wall. Do not stop until the forms are filled. At the proper points insert an overflow opening (if desired) and a sewer pipe connection for the water from the downspout or the filter and the pipe connection for the pump in the house. This pipe should be carried 3 ft. to 4 ft. underground, out of danger of freezing.



Concrete Cistern Platform with Manhole Cover

To keep out mice, vermin, scrub-water, or filth, every cistern should be covered with a concrete platform reinforced with steel rods. After the concrete walls have been brought to ground level set a wooden frame around the outside of the cistern, so as to hold the concrete for the 5-in. platform. To run water off the finished cistern cover, slope this frame 1 in. in the direction desired. Five inches below the finished top of the cistern cover construct a strong wooden platform of 1-in. boards for flooring. Build this on 2-in. by 4-in. joists nailed to the uprights of the side forms. This wooden platform will support the concrete cover until it is self-sustaining.

To provide for a manhole opening take 1-in. boards 6 in. wide and build a bottomless box 5 in. deep, 2 ft. square at the top and 18 in. square at the bottom—outside measurements. Grease the manhole frame and set it on the platform where the opening to the cistern is desired. An iron manhole frame and cover can also be used. By a similar use of a section of a gas pipe or small drain tile, leave an opening for the pump stock, if an out-door pump is to be used. Around this opening in the soft concrete place bolts (washered) for the pump. To locate the bolts correctly, set them by means of a wooden block in which holes have been bored spaced exactly like those in the iron pump base.

Pump Set on Concrete Cistern Platform.

Over the entire platform spread 1 in. of concrete. For reinforcing quickly place on this concrete 10-ft. lengths of $\frac{3}{8}$-in. iron rods running in both directions (criss-cross) and spaced 1 ft. apart. Bend the ends to a hook shape. Strengthen the platform around the manhole opening by placing two rods on each side. Bring the cover to its full thickness by immediately tamping in the remaining 4 in. of concrete. Finish the surface with a wooden float (a trowel) the same as for sidewalks. Reinforce the cover with short lengths of iron rods laid criss-cross. As a lifting ring for the cover, use half of an old bridle bit or a hitching post ring, the end provided with a knob of twisted wire or with a nut and large washer for holding it firmly in the concrete. If the wooden manhole form is used, carefully remove it after five hours. After three days build the manhole cover the same as for the tin form, with this exception—place greased paper or cardboard around the edge of the opening to prevent the new concrete from sticking to that of the platform.

To make the manhole cover lighter in weight, before placing the concrete spread 1½ in. of wet sand over the wooden platform in the manhole opening and then tamp in the concrete. Take care to place the reinforcing within 1 in. of the bottom of the manhole cover. When the cistern platform is two weeks old remove the manhole cover and saw an opening in the wooden floor. Descend, knock out the 1-in. wedges under the side-wall forms, take the forms apart and pass them through the manhole opening.

It is generally advisable to filter rain water. For this purpose provide a filter on the outside of the cistern proper. Construct this filter in the same way as the cistern was built, except that the reinforced concrete cover should be loose so that it can be removed. Therefore mould it on a smooth surface, such as a wooden floor, and later put it in place. Make the concrete walls and bottom 6 in. and the cover 4 in. thick.

Under most conditions a filter 2 ft. square and 3 ft. deep in the clear is large enough. Carry the walls a few inches above the surrounding ground. At the filter floor level connect the filter with the cistern by means of a 6-in. to 8-in. sewer pipe or drain tile laid with carefully cemented mortar joints. Likewise, 1 ft. from the top lay a similar connection for the pipe leading to the downspout. Instead of using a square inside form for the filter a barrel may be used. In the filter place a galvanised screen of ¼-in. mesh over the pipe opening to the cistern. Fill in 18 in. of coarse charcoal. Cover the charcoal with 6 in. of sand and gravel. Place the slab cover on the filter, and the cistern is ready for use.

THE REVISED REPORT OF THE ROYAL INSTITUTE OF BRITISH ARCHITECTS.

By R. W. VAWDREY, Assoc.M.Inst.C E.

While we do not associate ourselves in any way with the views put forward in this article, we think they are worthy of particular attention, and we shall gladly open our columns to correspondence on the subject. We would also particularly invite the views of Messrs. Hennebique on this important matter.—ED.

The revised report of the Royal Institute of British Architects has been dealt with at length in most of the technical journals and has also been commented on in the publications of the various reinforced concrete specialists, such as *The Indented Bar Bulletin* and *Ferro-Concrete.*

The matter is of such importance that a good deal of attention will naturally be given to it by those publications which deal specially with the subject. I wish now, however, to direct attention not so much to the actual report itself as to the above-mentioned comments upon it, particularly those of *Ferro-Concrete,* a publication devoted to the review of reinforced concrete construction in accordance with the Hennebique system.

The extreme agitation displayed by the last-named paper at the probable adoption by the London County Council of rules similar, if not identical with, those of the R.I.B.A. report is really amusing, but the attitude taken up is so dangerous that it cannot be altogether ignored. I hold no brief for the rules of the R.I.B.A. exactly as they exist at present; in fact, there are several points in which I think they might be altered with advantage, and I should very greatly regret to see anything like a tendency to restrict or hamper in any way original and progressive design, but the principle which I am now concerned to uphold is that of intelligent design on a rational basis, as opposed to rule of thumb design by means of empirical formulæ.

It is universally admitted by those to whom the safety and stability of reinforced concrete structures are of primary importance that in this material, as in all other forms of engineering or architectural work, the stresses which may possibly be induced in any member of a structure should bear some fixed and definite relation to the strength of the material of which that member is composed, and the main object of any set of rules issued by an authoritative body such as the London County Council is, of course, to provide a ready means of ascertaining before any structure is erected under its jurisdiction that the stability and safety of the structure will not be endangered by undue encroachment upon the factor of safety which it is desired to provide.

It is recognised that any such structure may be apparently quite satisfactory; that is to say, that it may have been completely erected, and may stand without at first showing any signs of failure, and yet that as a matter of fact the factor of safety may be ridiculously low, and the structure, although just able to carry itself under ordinary conditions, may all the time have so little reserve strength (*i.e.*, so small a factor of safety) that it may really be in danger of a collapse whenever any load at all beyond the ordinary is brought to bear on it, or when any conditions may obtain which are more than usually severe. It is just these conditions which are intended to be covered by the factor of safety specified, and it is the function of the L.C.C., and similar bodies, to ensure by their rules that a sufficient factor of safety is provided to cover any such—perhaps unusual, but certainly possible—conditions.

But *Ferro-Concrete* casts great scorn on "theoretical rules." One would almost imagine that the rules of design adopted by the Hennebique system were the product of accident, and had no rational basis. It is even stated that "theory is often opposed to fact." Nothing, of course, could be more grossly untrue if by theory is meant a correct theory, and such a statement is a contradiction in terms.

It is to be feared, therefore, that the Editor of *Ferro-Concrete* is convinced that the Hennebique rules admit in practice of a structure being submitted to larger stresses than would be permissible under the R.I.B.A. rules—or, in other words, that the factor of safety under Hennebique rules is less than that which would be ensured by the R.I.B.A. report. Why, otherwise, should it be desired to avoid compliance with the R.I.B.A. rules, which by pretty general consent are as satisfactory as can reasonably be expected, and have been adopted without demur by the great majority of those engaged in reinforced concrete designing in this country?

It may here be remarked that the implied statement that the R.I.B.A. rules have led to failures in this country since 1907, while the Hennebique rules have not done so, is not in the least in accordance with the facts.

Far more structures, designed on a great variety of systems, but all in accordance with R.I.B.A. rules, have now been erected in this country than those in which merely empirical formulæ have been used. Such formulæ are based on no rational theory, and, although they may prove satisfactory in one instance, they may subsequently prove by disastrous experience to be quite unsuitable for the next case to which they are applied. Surely by this time it should be unnecessary to state that rule of thumb methods are quite out of date in modern engineering.

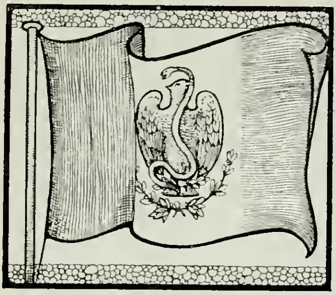
After all, the use of an empirical formula without a rational basis is a confession of ignorance, and it is surely obvious that reinforced concrete is the last material which ought to be treated by such rule of thumb methods as are apparently advocated by the paper in question.

It would be impossible to conceive a more effective argument in favour of the speedy issue of authoritative rules of design by the L.C.C. and similar bodies than the suggestion which would appear to be implied by the article referred to, that the Hennebique rules do not comply with the requirements of the R.I.B.A. report as now revised.

Very similar remarks are applied by the January issue of the Hennebique organ to the proposed regulations to be issued by the London County Council, and the objections made in this case, also, are no doubt due to the same fear that the insistence upon definitely limited stresses in the materials employed will be disadvantageous.

When, however, attention is turned to individual clauses instead of to the general policy of the regulations, I am in complete agreement with *Ferro-Concrete* on, at any rate, one point—namely, the objection which is raised to the portion of the rules dealing with "pillars." While I do not go so far as to agree that designers will be "forced into the employment of the least reliable form of transverse reinforcement," I do strongly urge the modification of the proposed severe handicapping of all methods of transverse reinforcement in columns other than spiral wires.

Why it should apparently be considered by the London County Council that spiral reinforcement so much transcends circular links, in which the same amount of metal is used, it is exceedingly hard to say. In my opinion, at any rate, the advantage in strength is all the other way, while in the question of cost, simplicity and ease in erection I suppose that even the advocates of spiral reinforcement would hardly claim that it competed in any way with separate hoops or links. In the case of a continuous spiral one weak spot affects the whole, whereas when separate hoops or links are used an irregularity or weakness in one does not in any way affect the strength and stability of the others. The difficulty, too, of getting the binding in close contact with the vertical bars is much greater in the case of a spiral than with separate links or hoops.

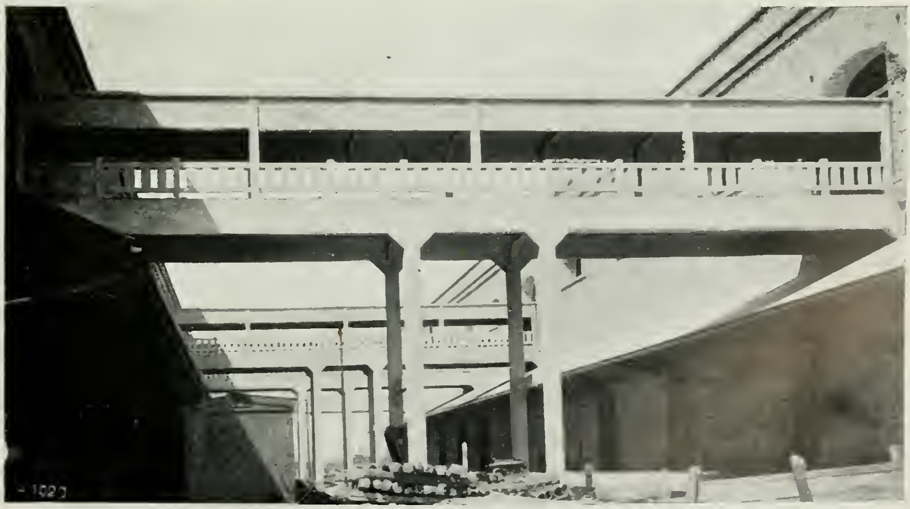


REINFORCED CONCRETE WORK AT THE RAILWAY TERMINALS, VERA CRUZ, MEXICO.

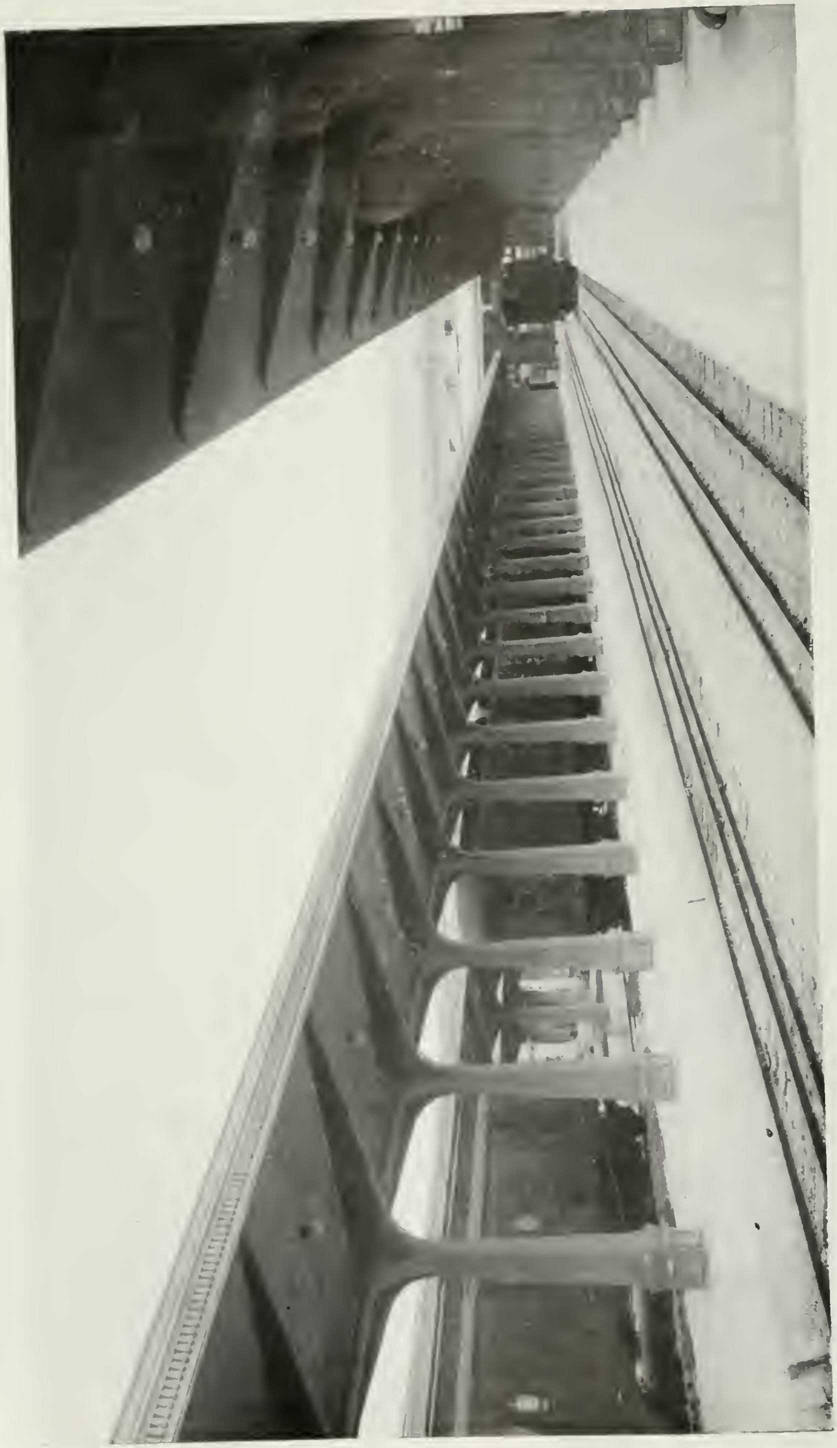
In our July, 1911, issue we gave particulars of various concrete and reinforced concrete structures in Mexico, and it affords us great pleasure to be able to show in the current number further examples of the rapid development of these materials in that country. We are indebted to Mr. R. W. Whitaker, Assoc.M.Am.Soc.C.E., for the photographs and particulars of these works.—ED.

IN April, 1906, a company composed principally of English stockholders was granted a concession for the handling of all imports and exports through the port of Vera Cruz, and in order to handle cargo in the most efficient manner it has been necessary to install modern terminal facilities. Construction work was begun in March, 1909, and completed in August, 1911. This work involved the expenditure of about twelve millions of dollars (Mexican currency) and comprised the construction of yard tracks, custom warehouses, a union passenger station, engine-house, shops and other necessary structures that are essential for a complete terminal.

Although a few of the buildings are of masonry and steel construction, it was deemed advisable to build most of the structures of reinforced concrete, because of the excessive corrosive action of the atmosphere in this climate.



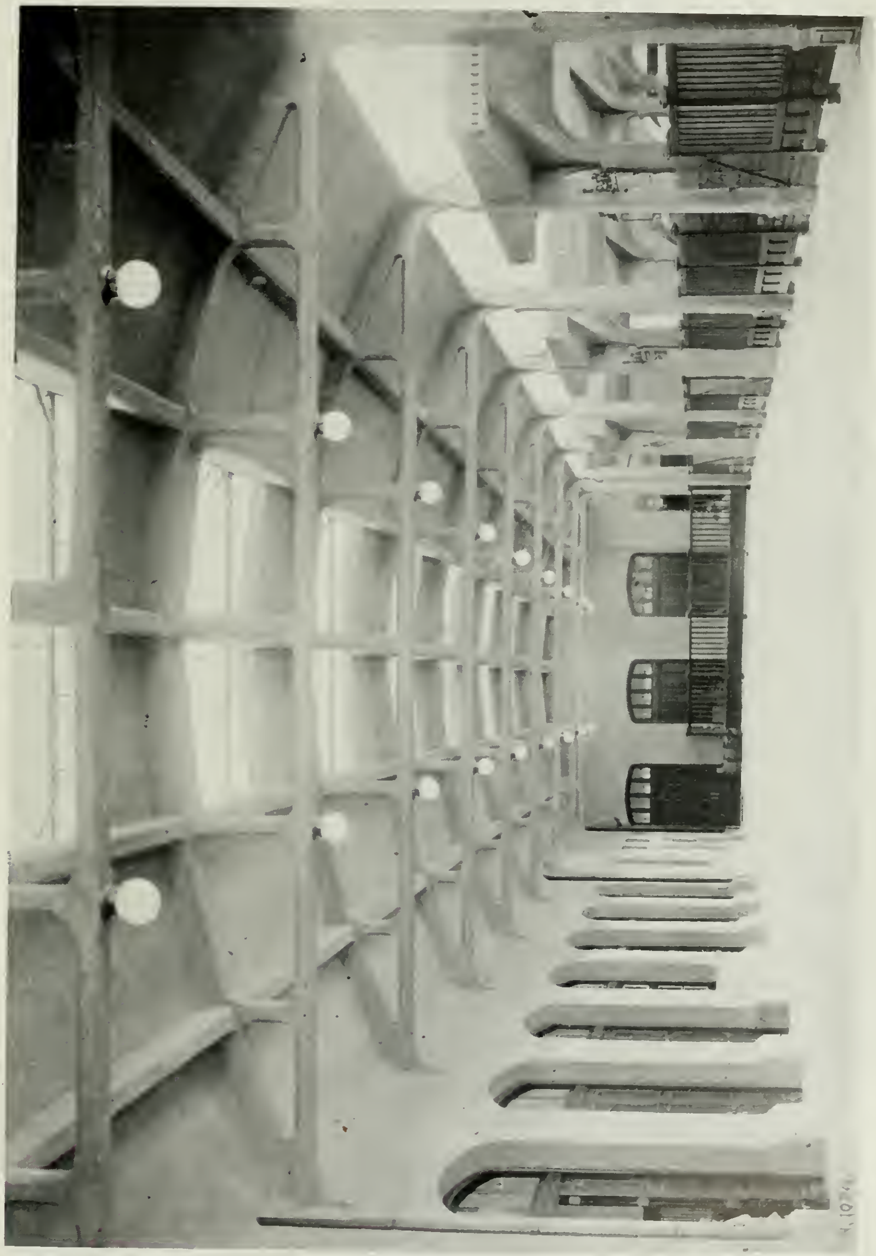
Reinforced Concrete Bridges between Customs Warehouses.
REINFORCED CONCRETE WORK AT THE RAILWAY TERMINALS, VERA CRUZ.



Passenger Platforms built of Reinforced Concrete.

Reinforced Concrete Work at the Railway Terminals, Vera Cruz.

The roof over the interior of the station is supported by concrete trusses of 41-ft. span spaced 21 ft. 4 in. centre to centre. The thrust of the top chord over the supports is taken care of by means of steel shoes which are anchored to the steel bars of the lower chord. The concrete used in the construction of the



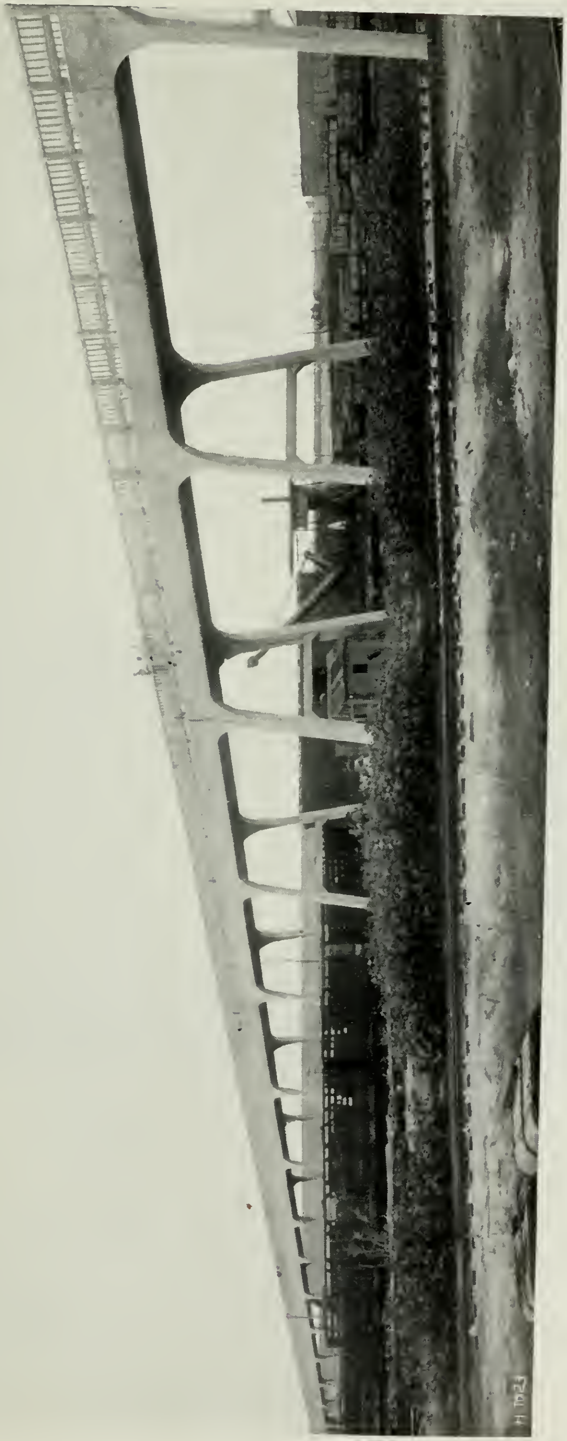
Interior of Station.

Reinforced Concrete Work at the Railway Terminals, Vera Cruz.

Reinforced Concrete Engine House.
Reinforced Concrete Work at the Railway Terminals, Vera Cruz.

trusses was of a 1 : 2 : 3 mixture. All joints were made in the roof beams midway between trusses. The lower chords were given a camber of 2 in. in order that there would not be any apparent sag of the trusses. The trusses and ceiling beams were given a bush-hammer finish, while a cement wash was used on the ceiling panels so they would conform to the exterior walls of the station. The skylights were placed in the roof in order to provide ventilation and additional light to the waiting-room of the station.

Over the four passenger platforms there are umbrella roofs, each of which is 493 ft. long and 23 ft. wide, supported by columns spaced 16 ft. apart. The roofs were designed to carry dead load only and to resist the wind pressure of the "Northers," which occur nearly every week during the winter season. From the nature of the design it was necessary to embed the column steel in the foundation concrete so that they would be securely anchored. The roofs are drained by means of 4-in. cast-iron down



Reinforced Concrete Foot-bridge, 993 ft. long.
REINFORCED CONCRETE AT THE RAILWAY TERMINALS, VERA CRUZ.

spouts placed in every other column. Most of the forms were used three times, but those for cornice and column capitals had to be partly rebuilt every time they were erected.

The bridge over the Terminal Company's property is 993 ft. in length, and is for foot traffic only. Most of the bents are spaced 52 ft. apart, so that there would be clearance for four tracks between bents. Although the variation of temperature in Vera Cruz is not more than 40° it was thought advisable to build two expansion joints to provide for variations in length, which otherwise might cause injurious cracks in the structure, specially in the railing, where the reinforcement is very light.

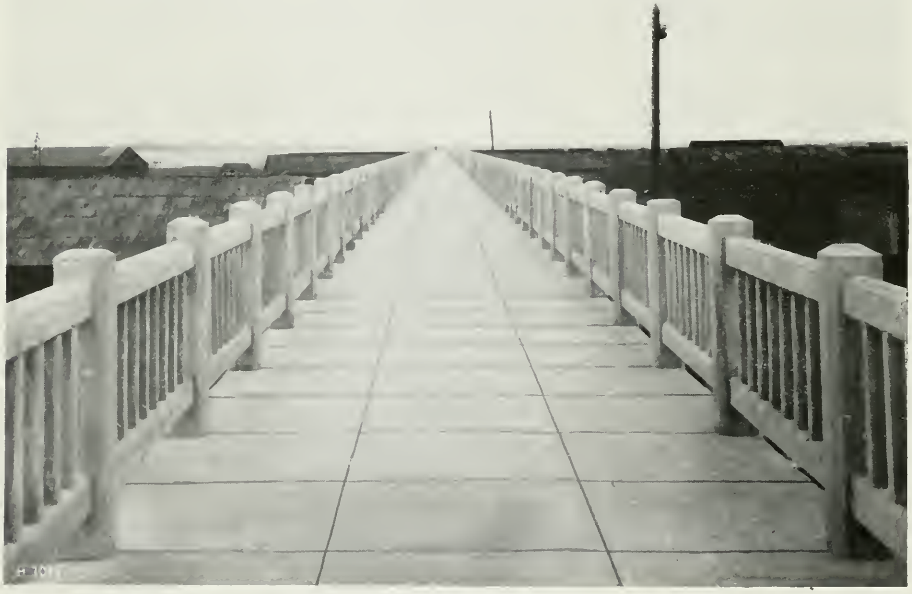
The railing shown in the illustration was adopted on account of being an economical type and of harmonising with the main part of the structure.

The top and bottom rails, together with the uprights for one panel, were moulded on the ground and were afterwards set in the forms for the railing posts, which were poured in place. The panels are 6 ft. 6 in. in length and light enough in weight for three men to handle them without having to use hoists of any kind.

The engine-house is a concrete structure having curtain walls of soft sandstone. There are twenty-four stalls, twelve of which contain pits for standard gauge engines, while the remaining twelve stalls are for the use of both standard and narrow gauge.

The four connecting bridges between custom warehouses were designed for a loading of 200 lb. per sq. ft., and also for a track loading of push cars carrying 4-ton loads. They are also built of reinforced concrete.

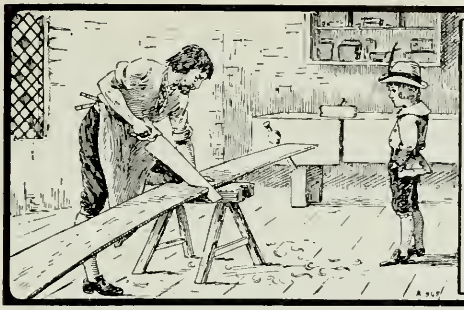
Although the stone and sand used in this work were the best that could be obtained within eighty miles of Vera Cruz, they did not possess all of the desirable qualities for good concrete work, as the stone was rather smooth, while the sand particles were very rounded, and considerable difficulty was experienced in keeping the materials from separating after the concrete had been mixed.



Reinforced Concrete Foot-bridge.

REINFORCED CONCRETE AT THE RAILWAY TERMINALS, VERA CRUZ.

All the work described above was designed by Mr. R. W. Whitaker.



CONSTRUCTION OF FORMS IN REINFORCED CONCRETE WORK.

By E. R. MATTHEWS, A.M.Inst C.E.

In our last two issues we published articles dealing with methods of constructing the forms for use in reinforced concrete work, and as this is one of the matters which requires serious attention in order that the greatest possible economy may be effected, we are giving a further contribution to the subject in the present number.—ED.

THE three matters that require the greatest care in carrying out works in reinforced concrete are:

(*a*) The construction of the forms and centering.
(*b*) The placing in position of the reinforcement.
(*c*) The mixing and placing of the concrete.

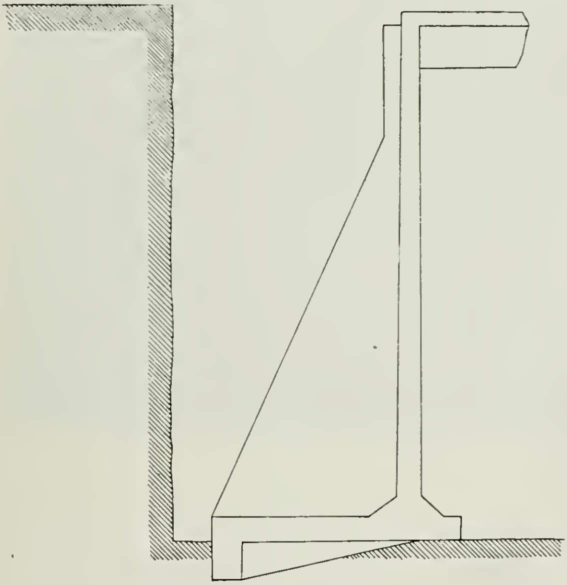


FIG. 1. OUTLINE OF RETAINING WALL.

The reader may at first think that these items include everything that requires attention in carrying out works in this material, but upon reflection it will readily be seen that many other matters, a dozen at least, have to be attended to.

The present article deals with the erection of forms so far as they are necessary in the building of retaining walls, and this is illustrated by reference to a retaining wall recently erected under the author's supervision at Bridlington by the Trussed Concrete-Steel Co. *Fig.* 1 is a cross-section through the retaining wall referred to. *Figs.* 2 and 3 show the construction of the forms, while *Fig.* 4 is a view of the finished work taken from the back of the wall.

Method of Procedure.—The erection of the forms should be proceeded with as follows:

The line of the wall having been most carefully set out, and the foundation slab put in, the erection of the uprights should next be proceeded with; there will be two rows of these, one inside, the other outside the wall. In order that these may be in line a cord should be fixed indicating the face of the finished wall, and the uprights on the outside of the wall should meet that line within, say, 1¼ in., which is the space necessary for the shuttering, assuming this to consist of, say, 9 in. × 1¼ in. boards.

The uprights should not be too light; if the wall is from 6 to 14 ft. in height (at Bridlington it varied from 10 to 13 ft. in height) 7 in. × 2 in. uprights should be used, and these should be spaced not more than 2 ft. 6 in. apart, or 5 in. × 2 in. spaced not more than 2 ft. apart. The uprights thus placed should be held in position by one or two rows of horizontal flooring 7 in. × 1 in. or 9 in. × 1 in. being nailed to same (see *Figs.* 2 and 3), and by numerous struts. The latter need not be heavy; 4 in. × 3 in., or even 4 in. × 2 in., struts will do nicely. The shorter ones may even be 5 in. × 1½ in. The top portion of the wall should be strutted independent of the middle portion in high retaining walls—that is, walls over 8 ft. in height.

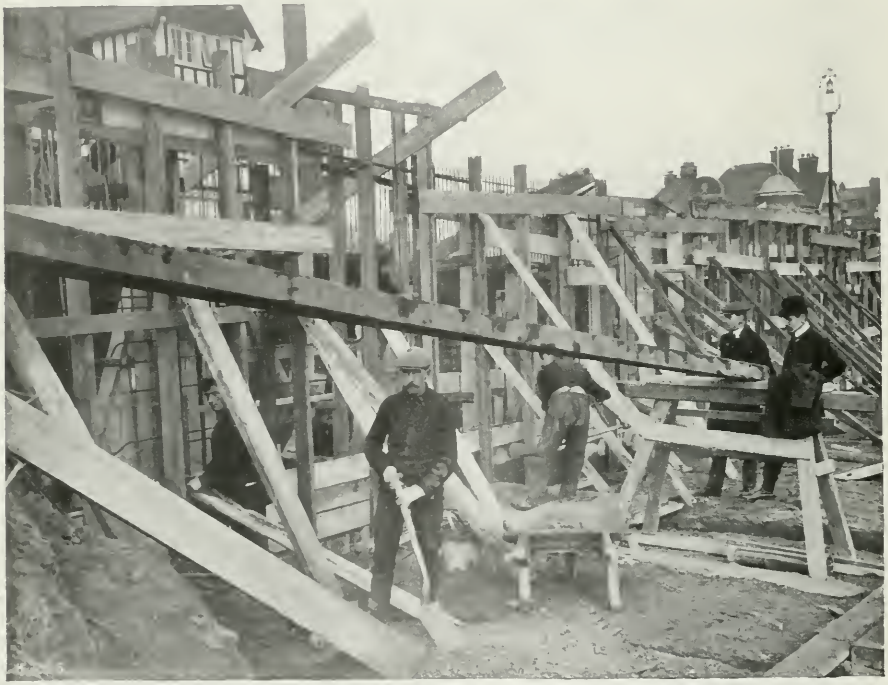


Fig. 2. View showing Uprights and Struts in front of Wall.

The uprights at the face of the wall having thus been well fixed and stayed, the first two, say, 8 in. × 1¼ in. boards forming the forms should then be fixed on the wall side of the uprights.

The supports at the back of the wall should then be proceeded with, and these will be fixed in a similar manner to those at the front, except that, if the wall is to have buttresses (as shown in *Fig.* 1), forms for these will require to be made and secured in their right places. They may be made of 5 in. or 7 in. × 1¼ in. flooring, and will be simply two sides of a box, being secured in front to two uprights, say, 7 in. × 2 in.

The upright flooring forming the forms should be secured to 5 in. × 1¼ in. diagonal pieces, as shown in *Fig.* 3.

In the Bridlington example the strutting at the back of the wall was different from that at the front, owing to the fact that there was the face of a cutting to strut from (see *Fig.* 1), the struts were therefore chiefly horizontal ones—namely, 5 in. × 2 in.

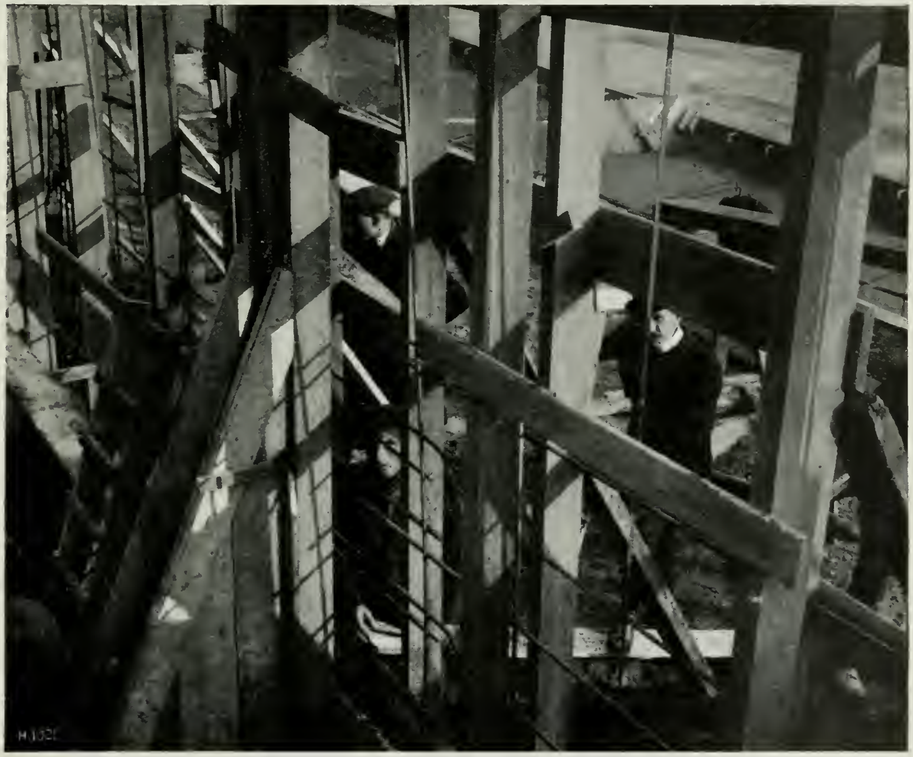


FIG. 3. VIEW SHOWING THE CONSTRUCTION OF FORMS FOR BUTTRESSES.

The forms for the buttresses should be boxed in, say, 18 in. ahead of the concrete laying, and so on until the top of the buttress has been reached. The boxing-in would be done by nailing short pieces of, say, 7 in. × 1 in. flooring on the slope.

As to the building of the wall, the concrete should be filled in in layers of not more than 6 in.; it should be fairly wet, so that it will require little ramming, all that is necessary being that it shall be worked into position and well around the reinforcement with a wooden tool about 3 ft. in length and 1½ in. in diameter; it is possible to get a special tool for this purpose.

A 1½-in. by ⅛-in. strip (w. iron) should be used to work the concrete in between the reinforcement and the face of the wall.

The forms should be well wetted inside and outside before any concrete is placed in them, and should be planed if the finished work is to be exposed to view; in fact, the extra cost of using planed boards is so little that the author would recommend planed boards in every case, as the finished work has a much better appearance.

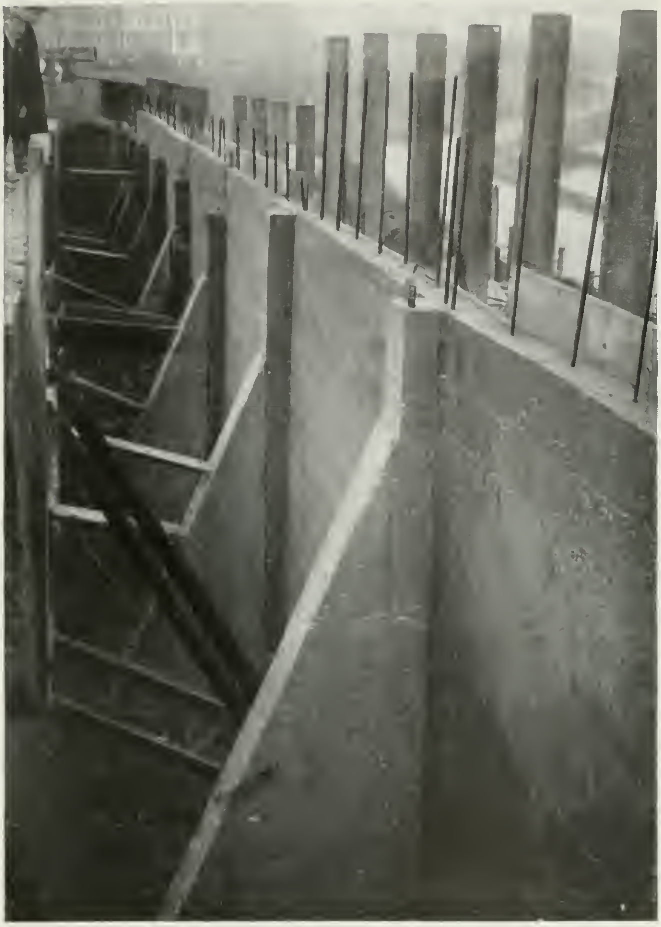


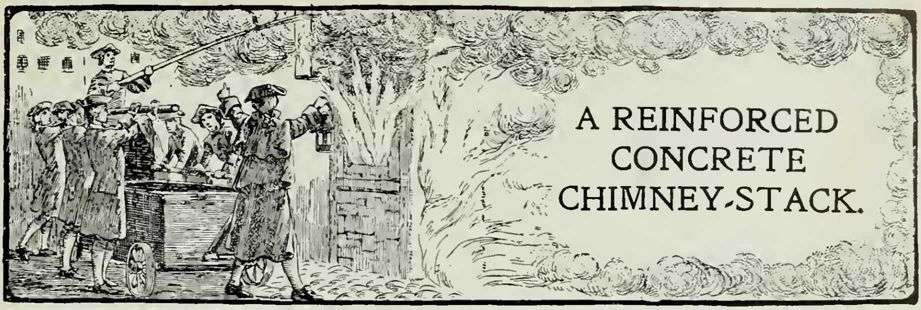
FIG. 4. VIEW OF BACK OF WALL TAKEN AFTER REMOVAL OF FORMS.

There is one little matter that should be noted regarding the fixing of the uprights when a fillet has to be made, as shown in *Fig.* 1. The author found that it was better to fix the uprights on the foundation slab, as already suggested, and then let the fillet be formed between these, boxes being, of course, made for same, the cavities made by the uprights being filled in with concrete after the 5-in. × 2-in. or 7-in. × 2-in. uprights had been removed.

Great care should be taken that the upper portion of the wall is well stayed in case of severe gales. A short length of the wall just erected by the author at Bridlington suffered slightly in its upper 3 ft. by two or three of the uprights having a little play at the tops.

He did not find it necessary to coat the forms with soft soap or vaseline to facilitate their removal. All forms were taken down at the expiration of about a month.

In conclusion, he would say that in his opinion much excellent reinforced concrete work is spoiled by insecure and insufficiently rigid forms and centering, and he urges those engaged in the carrying out of works in this material to pay greater attention to this important part of the work.



A REINFORCED CONCRETE CHIMNEY-STACK.

By OSCAR FABER, B.Sc., A.M.I.C.E.

The article below on the construction of a reinforced concrete chimney-stack is taken from an article by Mr. Oscar Faber in "Engineering," December 22, 1911, which should be of interest to our readers.—ED.

The reinforced-concrete chimney described below was erected to the order of a company in London on a site near Drury Lane, W.C., the object of the stack being to carry the exhaust gases from internal-combustion engines to such a height from the ground that they cannot constitute a nuisance in the surrounding neighbourhood. The stack is erected to replace four steel pipes having a total height of 85 ft. from the ground, which were found to be insufficient to effect the above-mentioned purpose.

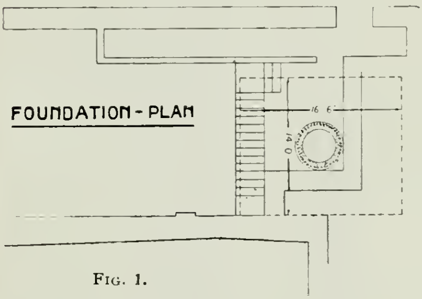


Fig. 1.

The new shaft is 144 ft. 9 in. in height, measuring from the top of foundations, which coincides with basement-floor level. As the volume of gases to be carried up is comparatively small, an inside diameter of 2 ft. 6 in. was considered sufficient.

It will be of interest here to explain the considerations which led to the erection of the chimney in reinforced concrete. The chimney, if built of brick, would have to comply with the London Building Act of 1894, Section 65, which enacts, *inter alia*—

(1) Every shaft shall . . . taper gradually from the base to the top of the shaft at the rate of at least 2½ in. in 10 ft. of height.

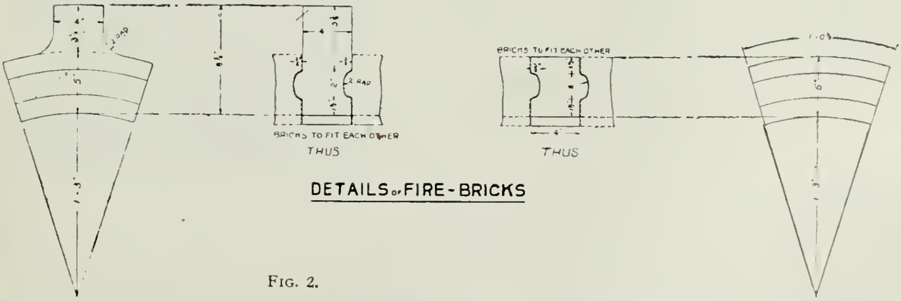


Fig. 2.

(2) The thickness of brickwork at the top of the shaft . . . shall be at least 8½ in., and shall be increased at least one half-brick for every 20 ft. measured downwards.

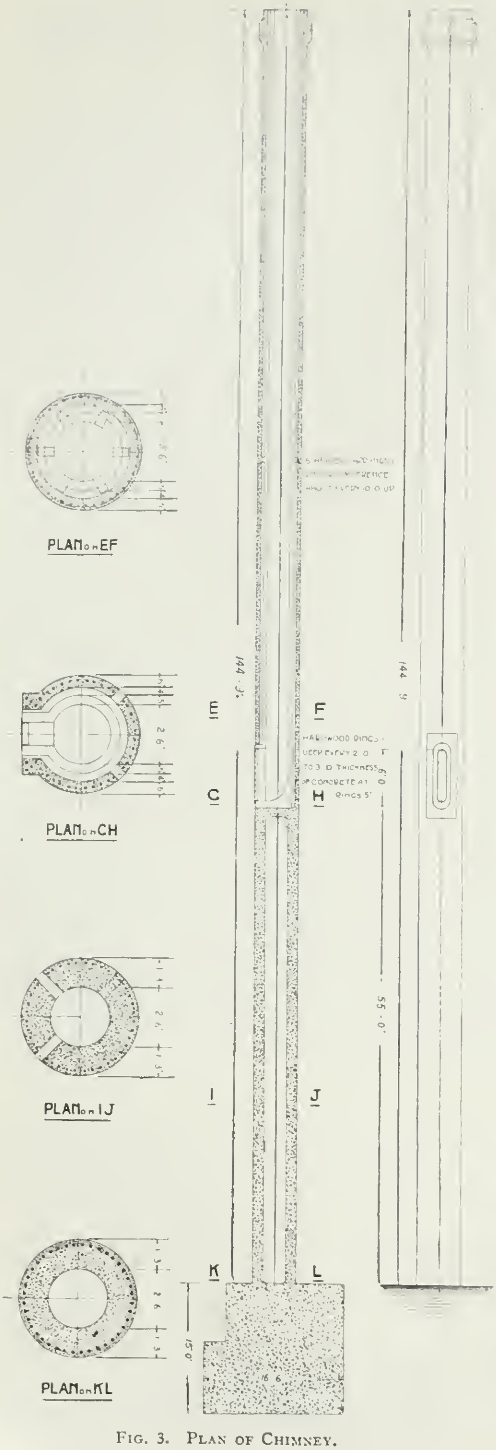


Fig. 3. Plan of Chimney.

(6) The width of the base of the shaft . . . shall be at least . . . if the same is round . . . one-twelfth of the height.

(7) Any fire-bricks built inside the lower portion of the shaft shall be provided as additional to, and independent of, the thickness of brickwork prescribed by those rules, and shall not be bonded therewith.

It will be seen from these that the minimum dimensions permissible for the brick-shaft would be:

Outside diameter at top	..	4 ft.
,, ,, base	..	12 ft.
Wall thickness at top	...	9 in.
,, ,, base	...	3 ft.

Allowing a wind pressure of 25 lb. per sq. ft. on the projected area of the chimney (this pressure being assumed to include the reduction factor for circular cross-section), it will be found that the overturning moment

$$M = 1{,}748{,}000 \text{ ft.-lbs.},$$

and the weight

$$W = 638{,}000 \text{ lb.}$$

Whence

the eccentricity $e = 2{\cdot}74$ ft.

If the brickwork were capable of resisting tension, the maximum compressive stress would be

$$C = \frac{638{,}000}{84{\cdot}75} + \frac{1{,}748{,}000}{158{\cdot}8}$$

$$= 7530 + 11{,}000 = 18{,}530 \text{ lb./ft.}^2, \text{ or } 128 \text{ lb./in.}^2$$

This figure would only be justified if the brickwork were capable of resisting a tensile stress of 3,470 lb./ft.^{2}

or 24·1 lb./in.^{2}

The exact calculation of the maximum pressure when the tensile strength is ignored is not a simple matter, although it may be determined in the same manner as that advocated for reinforced-concrete chimneys in the article by Messrs. Taylor, Glenday, and the writer, which appeared in *Engineering,* March 13th, 1908. It is obvious, however, that it would exceed 128 lb. per sq. in., whereas 125 lb. per sq. in. is the limiting stress allowed by the London County Council for hard brick in cement mortar. It will be seen, therefore, that an outside diameter of 12 ft. is not only the least permissible by the London Building Act, but is also the least which a prudent engineer would care to erect.

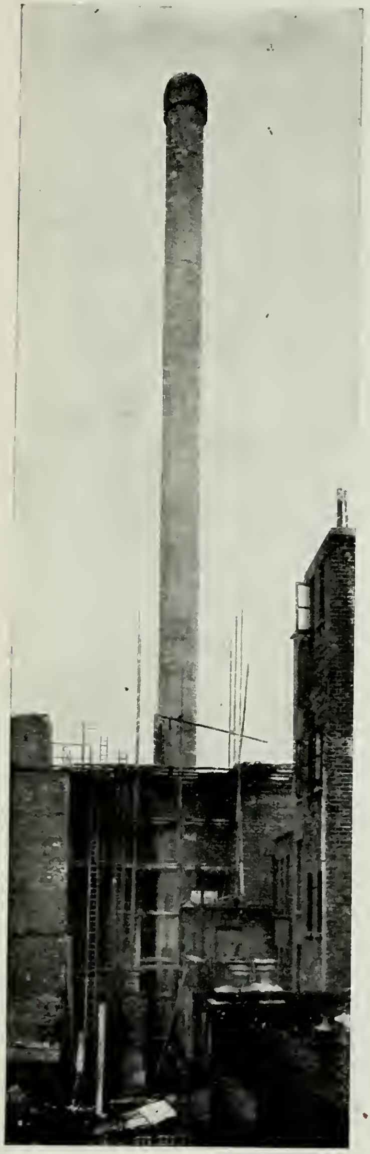


Fig. 4. View in course of construction.
A Reinforced Concrete Chimney-Stack.

This size of chimney, though small in comparison to its height, is, however, considerably greater than could be erected at Short's Gardens, owing to great scarcity of room. The alternatives were therefore reduced to a self-supporting chimney of steel or reinforced concrete. The first of these could not be passed as a permanent structure by the London County Council, owing to the corrosion of the steelwork, and only a reinforced-concrete chimney would meet the case. When it is seen that the diameter at the base was kept down to 5 ft., it will be seen how effective was the substitution of reinforced concrete for brick.

The foundation plan is shown by *Fig.* 1, while the details of the design are illustrated in *Figs.* 2 and 3. From these it will be seen that the lower portion of the chimney, up to flue entrance, is 1 ft. 3 in. thick, and above that 6 in. thick. Above the flue entrance a lining of firebrick 5 in. thick is provided, with an air-space 4 in. thick between it and the outside shell. Vent-holes are left near the flue entrance, through which a current of air is induced in the air-space between the liner and the outer shell.

Some theoretical calculations of temperature stresses led the writer to a system of construction in which timber rings are embedded at intervals on the inside face of the concrete shell, which practically reduces to zero the stresses due to expansion. This system of construction, which is patented by the writer, was adopted by the Associated Portland Cement Manufacturers in a chimney built by them at their works at Burham, on the Medway. This chimney has now been in use several months, and is quite free from any cracks due to temperature stresses. The chimney forming the subject of this article was also built on this system, and is here also proving quite successful.

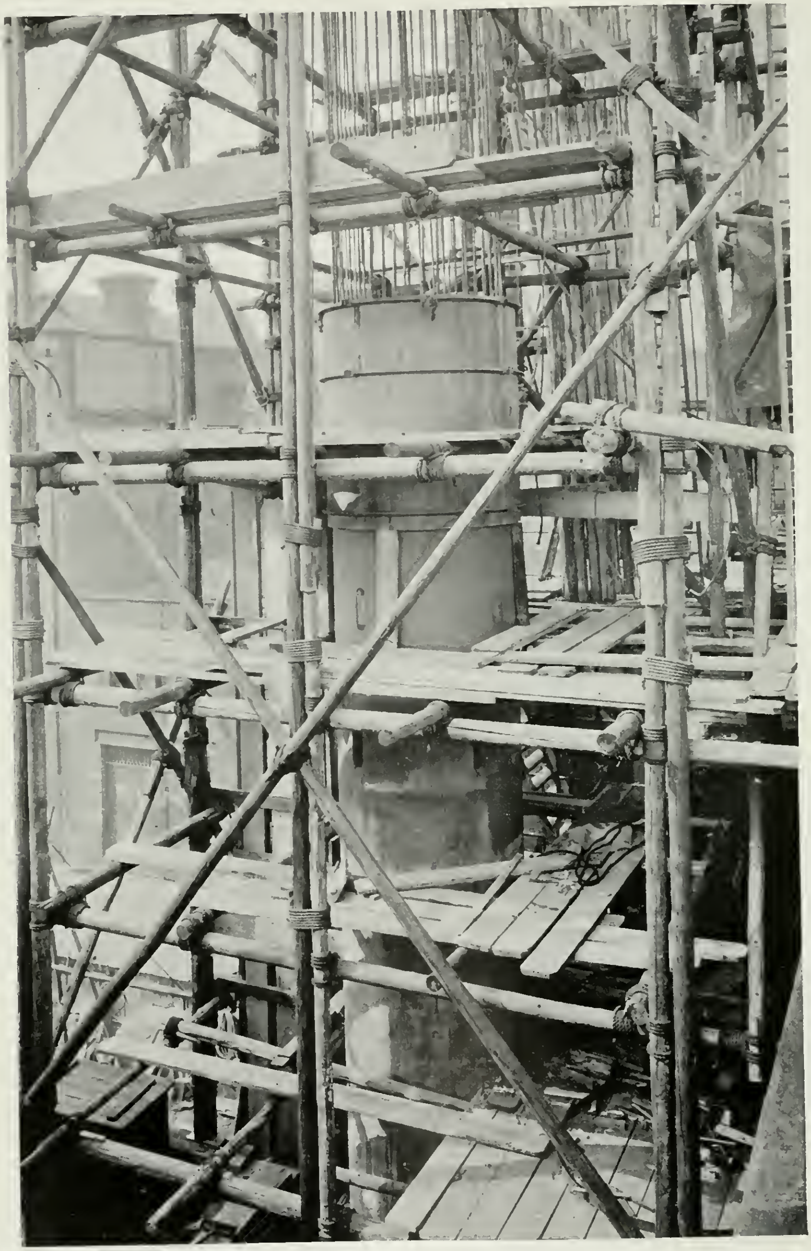


Fig. 5. View of reinforcement in place and scaffolding.

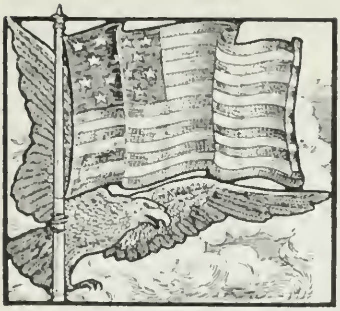
A Reinforced Concrete Chimney-Stack.

The foundation had to be taken down 15 ft. below basement level, which is itself 14 ft. 9 in. below ground level, and as it was necessary to underpin the existing walls under which it was built, it was made a solid block of concrete 16 ft. 6 in. by 14 ft., thus serving for the underpinning also. A staircase is formed out of the foundation block for giving access to the engine-room, which is below the yard.

The reinforcement used in the chimney were indented bars, supplied by the Indented Bar and Concrete Engineering Co., Ltd.

The firebrick lining consists of special firebricks made with a raised rib, 2 in. by ¾ in., fitting into a corresponding recess in the next course. To ensure the lining remaining concentric with the outer shell, special bricks are used at intervals 10 ft. apart, with side projections. About ⅜-in. clearance is left between these projections and the outer shell to allow for expansion of the firebricks.

The work was carried out by Messrs. Trollope & Colls, of 5, Coleman Street, E.C., to whom the writer is chief engineer.

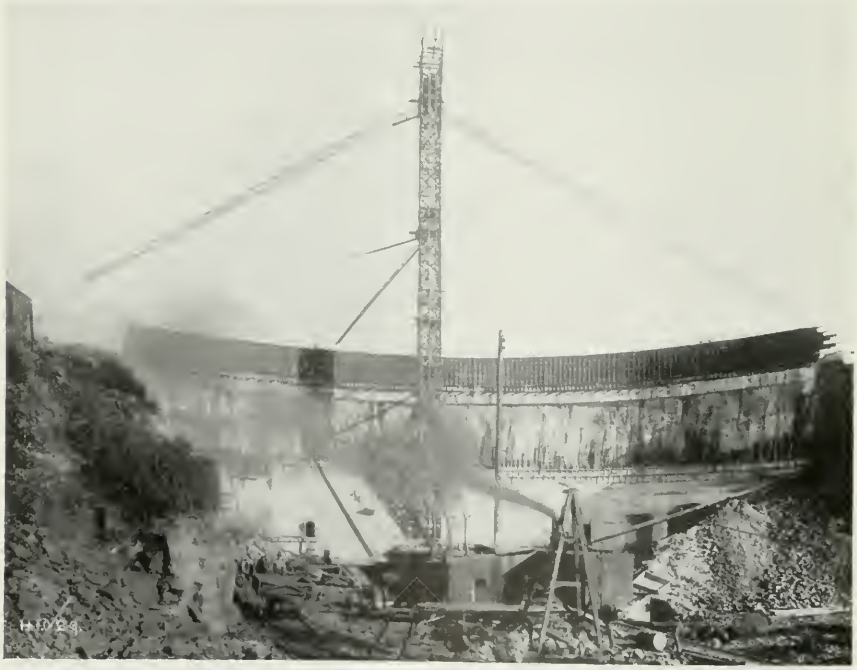


THE EXTENSION OF THE SWEETWATER DAM, CALIFORNIA.

The wall described below, which consisted in the alteration of a concrete dam—which was built some twenty-three years ago—from the arch to the gravity type, contains some engineering features of special interest.—ED.

In the spring of 1910 the Sweetwater Water Company decided to enlarge the reservoir, which had been built twenty-three years previously, to about double its capacity by the addition of 20 ft. to the height of the dam.

An account of the interesting work entailed by this addition is given by Mr. James D. Schuyler, M.Am.Soc.C.E., who designed and supervised the

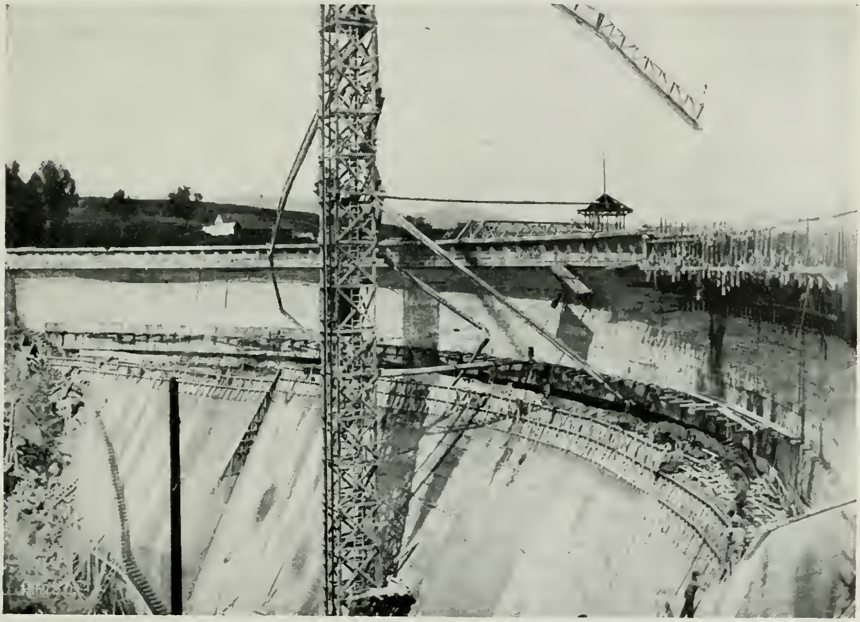


Tower for Distributing Concrete.
THE EXTENSION OF THE SWEETWATER DAM, CALIFORNIA.

addition as well as the original structure, in *The Engineering Record*, from which our particulars are taken.

The effectiveness of the completed dam must depend essentially upon the combined resistance of both the old and the new walls, and since the latter has to be regarded merely as a gravity wall, it was deemed best to ignore the beneficial influence of the existing curvature further than as a substitute for the reinforcement against tensile strains lacking in a straight wall.

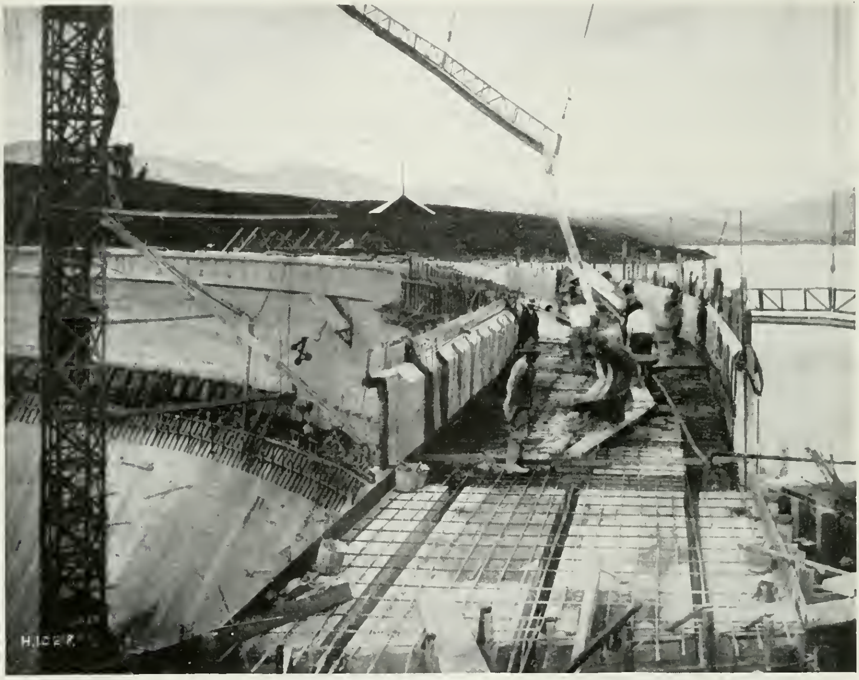
It was decided, therefore, to make the old dam capable of resisting by gravity alone and under moderate unit stresses the maximum load imposed by a water plane at El. 220·0. Second, to design the additional structure so that by its combination with the old one the finished dam might resist by gravity alone the maximum pressure caused by a rise of the lake to the crest of the new parapet at El. 240·0, or 5 ft. above the spillway.



View Showing Change of Section of New Wall.
Extension of the Sweetwater Dam, California.

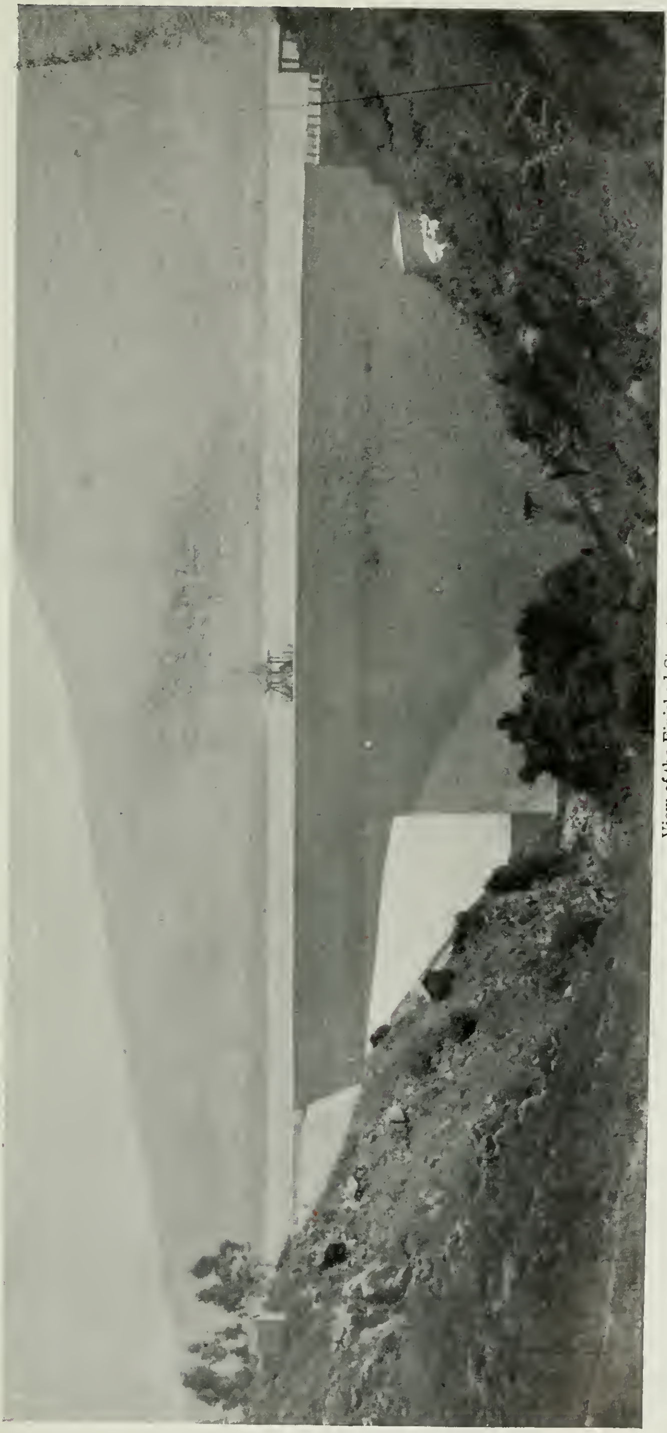
These results were, of course, to be accomplished with the greatest possible economy of masonry.

In the old wall, under the maximum load of full reservoir to the crest of its parapet, El. 220·0, the pressures were 31,564 lb. per sq. ft. at the toe and 11,550 lb. per sq. ft. at the heel. In fact, the negative pressure indicated meant a considerable tension, or about 80 lb. per sq. in. This was not serious in a masonry wall of the high quality of construction possessed by the old dam, but it is considered good practice to avoid tensile stresses in masonry, and in order to place the dam in normal condition as a gravity structure the one thing needed was to effect a better distribution of pressures over the foundation. This foundation, by the way, is a tough porphyry rock of many dips and angles, forming an ideal surface for bonding with masonry. To distribute the pressure so as to eliminate tension it was only necessary to remove the toe 13·25 ft. farther down stream, when the pressures became positive for any further increase of base. This extension of the toe of the old wall was, of course, a portion of the foundation of the new wall, in which the stresses of the old structure were transmitted to the foundations by a system of steel rail reinforcement. Since the use of tension reinforcement in the upstream face of the old wall was out of the question, the substitute for it had to be made by compression members on the downstream side. A grillage formed of three layers of 60-lb. rails laid horizontally was built up in the footing of the new foundation so as to insure its rigidity in spreading the pressure over the bedrock. This grillage was connected with the old dam in the following manner:



Form Work at the Top.
The Extension of the Sweetwater Dam.

A distance of 16 ft. was measured from the toe of the old dam as representing the width of the theoretical extension of its base, and in this space inclined tiers of 60-lb. track rails, 3 ft. and 3·5 ft. apart in vertical plane, and 3 ft. apart horizontally, were embedded in concrete with their lower ends resting on horizontal courses of single rails, placed concentric to the old dam



View of the Finished Structure.
The Extension of the Sweetwater Dam.

immediately overlying the bottom grillage. The upper ends of these struts were similarly capped by horizontal rails. The struts were inclined upward toward the downstream face of the old wall, at an angle with the horizon slightly smaller than the angle of repose of concrete on rubble. As the rock ledge is unyielding all pressures tending to crush the masonry at the toe are met by the steel and concrete pillar formed by the armoured portion of the footing with no possibility of sliding. In other words, the old wall is now braced against the bottom of the pit into which the new one is built by a toggle action whose only limit is the crushing resistance of the concrete and steel of the strut.

With its toe thus extended the maximum load on the old wall is distributed to the foundations as follows:

At the new toe, 13,542 lb. per sq. ft.; at the old toe, 10,380 lb. per sq. ft.; at the heel, 1,273 lb. per sq. ft. All the pressures thus become positive and tension is eliminated.

The new masonry consists of three distinct divisions:

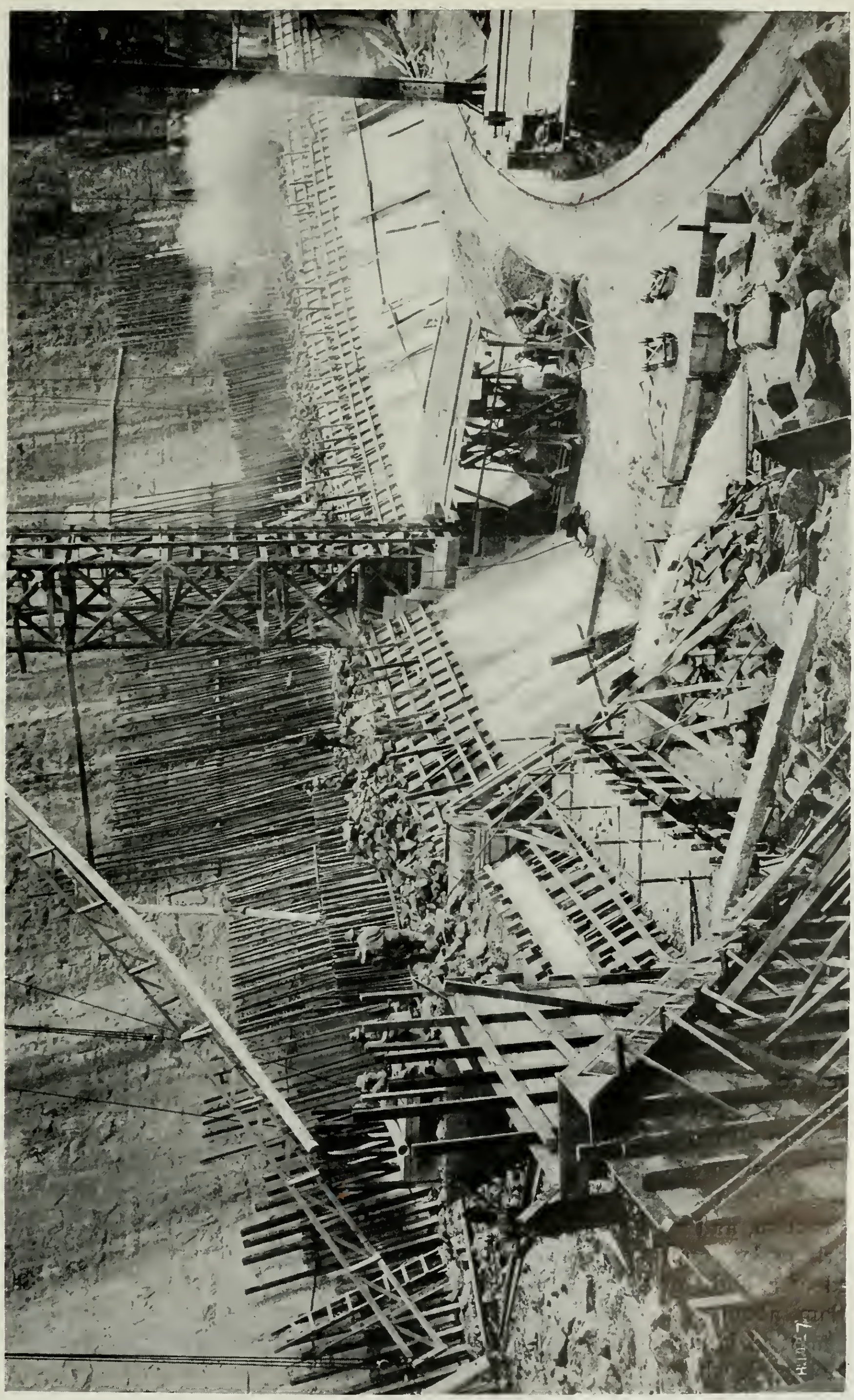
1. A wall of concrete in the form of a concentric monolith built against and resting upon the downstream face of the old dam.

2. A vertical extension of the old wall, with parapets carrying the roadway and surmounting the old wall.

3. An independent spillway channel, built parallel with the lake for 228 ft. and passing through the south end of the dam, where it connects with the old waste way, well fortified in 1895 by an apron of concrete and steel to receive the batter of any possible overpour.

The Downstream Wall.—The height of the new slab added to the downstream side of the old wall, its great weight and the comparatively short time in which it was to be built seemed to point to the certainty of material settlement during or shortly after construction, with consequent damage to the region of contact in the shape of molecular stresses and not improbably of more or less extensive cracks in case an attempt had been made to bind the new work to the old by anchor rods or by mere surface tension welding.

This was not, however, the most serious consequence to be anticipated and avoided. While the old dam had been fairly watertight and most of the original seepages had dried up after a long period of laitance action, there were some small leaks representing a constant outflow of water, although in reality of no import except as affecting the appearance of the structure. With a new wall closely joined to one oozing with water in places, the practical effect would, in time, have been to transfer the entire hydrostatic pressure from the old wall to the new one, instead of dividing the load between them. The new wall was, of course, not designed to substitute the old one, but only to reinforce it, so that unless the leakage could be absolutely disposed of the new wall might be ruptured by the imposition of unforeseen strains. It was, therefore, decided to forego any attempt to unite the new work to the old, but to purposely separate them by such spaces as will afford channels for the water to pass off permanently and freely without accumulating head against the new wall. To accomplish this object sheets of No. 27 corrugated steel were spread over the entire downstream face of the old dam in horizontal rows, with the usual overlap of 2½ in. customary in roof covering, the corrugations being set vertically. The horizontal seams, likewise overlapping, were covered with coarse muslin to prevent the mortar from oozing through the joints. This curtain of corrugated steel was carried up slightly in advance as the wall progressed and appeared to serve the purpose admirably, as each corrugation, reaching from bottom to top, forms a drainage channel. The water is drawn off from behind the sheet through 2-in. pipes placed at intervals of 10 ft., leading into a drainage tunnel 6½ × 3½ ft., built concentric with the old dam a few



Reinforcement of New Wall.

The Extension of the Sweetwater Dam.

feet away from it, with a branch leading outside for discharging the water entirely below the wall.

In placing the concrete against the corrugated plates every corrugation was thoroughly filled so that in effect the points of contact between the new wall and the old one consist of a series of continuous vertical ridges faced with steel $2\frac{1}{2}$ in. apart, with intermediate valleys or drains. The area of contact was sufficient to prevent crushing of the plates, which were, moreover, stiffened by the concrete filling in the corrugations. The lower tier of sheets rested on top of a small conduit extending entirely across the face of the dam, with a slight grade from each end towards the centre.

When the two walls are simultaneously under load—viz., when the water rises above El. 220·0—the distribution of pressures becomes as follows :

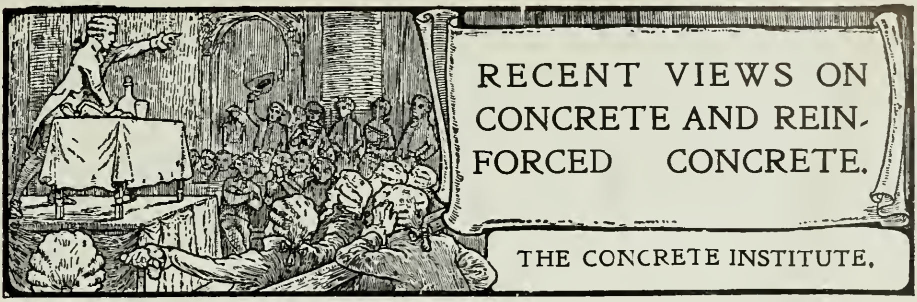
At the new toe, 11,651 lb. per sq. ft. ; at the old toe, 7,037 lb. per sq. ft. ; at the old heel, 0 lb. per sq. ft. ; and the resultant of pressures meets the foundation at the point of passage from its middle to its downstream third. The stability of the structure, both in separate parts and as a whole, is therefore insured.

In the determination of the resistance against overturning of the new wall, considered as a vertical beam, fixed at its lower end in the bedrock foundation by virtue of its dead weight and the cohesion of the concrete to the ledge, tensile strains are found to develop in its face of contact with the old dam greater than the ordinary working stresses used with plain concrete. Hence it became necessary to counteract these stresses by a reinforcement of 40-lb. and 30-lb. track rails, extending upward to a point where the concrete alone could safely withstand the bending stresses developed ; about 307·5 tons of rails were thus employed.

Crest and Parapet.—The portion of the enlarged dam which gives the actual increase in storage capacity sought rests on top of the old wall almost exclusively, and is actually independent of the great slab of concrete which is mentioned hitherto as the new wall. Its cross-section is practically a triangle with thin walls at intervals of 10 ft. carrying a 12-ft. roadway with parapets and a thin wall. The new crest wall has been anchored to the old wall with two $1\frac{1}{4}$-in. square twisted steel bars, 10 ft. long, placed at intervals of 5 ft., about half their length being inset in large drill holes side by side and the upper half being embedded in the new concrete.

All of the new concrete on the water face was covered with a coat of cement plaster, to which hydrated lime was added to the extent of 10 per cent. of the cement in the plaster. About 5 per cent. of hydrated lime was used, in fact, throughout all the concrete to obtain maximum density and watertightness.

The system of concrete delivery adopted by the contractor, Mr. Arthur S. Bent, of Los Angeles, was one which was developed in this city on building construction, and used at the Sweetwater Dam for the first time on so extensive a work as dam building. It proved to be a very satisfactory method. It consisted of a high tower, over 200 ft. in extreme height, erected at the toe of the dam, about at its centre. At the foot of this tower the concrete was mixed and emptied into a dumping skip, which was hoisted up to the tower to any desired level, where it was dumped into a hopper feeding into light sheet-iron pipes, or flumes suspended from a fixed cable.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

THE CONCRETE INSTITUTE.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE CONCRETE INSTITUTE.

THE AESTHETIC TREATMENT OF CONCRETE.

Paper by PROFESSOR BERESFORD PITE, F.R.I.B.A.

At the First Summer Meeting of the Concrete Institute held in June last Professor Beresford Pite delivered a lecture on " The Æsthetic Treatment of Concrete." In our issue of July, 1911, we gave a short summary of this paper and of the subsequent discussion. Owing to the importance of the subject, it was decided that another meeting should be held to discuss the matter further. This meeting took place on February 8th, 1912, Sir Henry Tanner, I.S.O., F.R.I.B.A., President of the Institute, in the chair. We are again giving the short abstract of the paper supplied to us by the Institute, together with a lengthy summary of the second discussion. A number of lantern slides were shown of various reinforced concrete structures, and the lecturer dwelt at some length on the particular features of each building.

SIMPLICITY of intention in constructional design may issue in a native or spontaneous æsthetic quality. An undesigned beauty reached without treatment is often attained by such a structure as the Forth Bridge or a reinforced-concrete silo. Mediæval architecture grew up as a constructive method without æsthetic purpose and achieved results of great beauty. Is not the opportunity given by the new process of reinforced concrete building one that could be utilised for the erection of the much-desired original and modern style of architecture? Are the new material and method together sufficient motive? Truthfulness of design to constructive purpose and elemental soundness of proportion, are these sufficient to provide that pleasantness to the eye which is desiderated?

Four conclusions arrived at: (1) We have no instinctive guidance towards an unbiassed originality for a concrete architecture; (2) Abstract principles like those invoked of proportion are of no assistance; (3) Superficial treatments, as by colour, are insufficient for architectural expression, though valuable in assistance; (4) The texture of concrete surfaces modifies and imparts special character to any forms employed for architectural purposes. Therefore, while modern considerations of utility and of novel constructional methods determine proportions, and may ultimately develop æsthetic qualities, the scholarly and critical analysis and employment of traditional architectural forms, 'suitably modified for execution in concrete, is the proper method for the æsthetic treatment of concrete.

A historical review of the development of some characteristics of Egyptian, Greek and Roman architecture furnishes proofs of the non-relation of æsthetic treatment to direct constructive facts. Idealised representations of ancient types form the basis of both Egyptian and Greek characteristics, while the Romans frankly separated the decorative from the practical purposes of architecture. In Gothic art, however, the constructive craftsman was the artist, and the development of decoration is integral with building craft. In the other crafts of wood and plaster work motives are imitated from stonework and illustrate the modifications produced by the texture of the material into the design of details: of this the Elizabethan ornamental plastered ceilings originated from Tudor vaultings are illustrations.

Modern novelty of constructive method does not remove necessity for study of architectural development, but it will aid adaptation and modification, and thus pave the way for development. Modern Continental design is too eager to demonstrate elasticity possible in employment of form in unusual architectural relations. At home we still are safely and timidly putting brick and stone fronts to concrete buildings. There is a great future before concrete building, and it deserves that close and patient architectural study which deriving from the past will give certainly to the future æsthetic treatment of the material.

ADJOURNED DISCUSSION.

Mr. A. Alban H. Scott, M.S.A., proposed a vote of thanks to Professor Beresford Pite for his attendance that evening to give them the chance of further discussing his paper.

It was a paper which, as a Concrete Institute, it was most difficult for any one of them to study, perhaps because none of them was qualified to discuss in any way a question of architecture or its treatment.

The line which he would very much advocate was that they should give up practically all idea at present of applying ornament to reinforced concrete work, because he felt that if they attempted to apply ornament at the present time they would get most disastrous results.

But with this new material there were other problems which arose which in every way altered the constructional work. There was one point which had been brought up quite recently —the treatment of the under-surfaces of floors with very large spans. In every building of reinforced concrete it would be necessary to introduce in the constructional parts beams of greater depth than the floor-covering material; but there was a type of floor, now known as the mushroom type, where it came up and spread out in a sort of fan-shape, and the whole of the factory was flat. He thought that in a very large building, a factory or otherwise, an absolutely flat ceiling without any beams. for the moment, gave one rather a shock; but he thought that was a proper and efficient treatment of this material, though perhaps it would not conform to the grammar of architecture.

He had asked an engineer that evening if he thought it was possible to get successful façade treatment in reinforced concrete, and he said most decidedly. It turned out that it was treatment of plaster inside jointed up to imitate stone, which was a thing he thought they should avoid; and that was one of the reasons why he advocated no ornamental work for concrete. They should leave it exactly as it came from the centering and only take off the large excrescences and leave the work as if it was hand work as opposed to machine work.

He would go farther than that, and would not have the concrete work faced up absolutely to a dead line. He did not say he would eschew concrete in work of that sort, but he did not attempt to get a very definite line anywhere.

Some people had an idea that concrete was an essentially coarse material, and a material which required coarse treatment: but he was very much opposed to that. He liked to think that it ought to be treated with the highest degree of refinement. He would do the same with concrete as with cast-iron material, and would not attempt to trim off every little point that happened to come along. It was rather unfair to ask whether Professor Beresford Pite would consider that it was a legitimate thing absolutely to construct the vaults of a church with reinforced concrete. In that case he presumed that the lecturer would not be opposed to superficial treatment such as is likely to be caused by stonework. That was a case where, for interior work, superficial treatment, mosaics or paintings, were an essentially correct treatment, and you could give it, if necessary, a coating of plaster or cement.

Mr. Arthur T. Bolton, A.R.I.B.A. — There was a question whether reinforced concrete was known to the Romans or not. He would suggest to his engineering friends to-night to consider the cold-water baths of the Baths of Caracalla at Rome. Now this was a gigantic hall with a span of 60 ft. or 80 ft., or something like that. and ancient writings told them that its flat ceiling was one of the marvels of Rome. The ceiling being flat, it had in it three or four large openings for lantern lights. How did the Romans construct a concrete ceiling on that gigantic scale flat and with these openings in it? Professor Lanciani, who has devoted a lifetime to the study of antiquities of Rome, mentioned in one of his books that when this cold-water bath was explored great quantities of T-irons were removed. The plan of Caracalla's Baths was in every textbook on architecture. Now our engineering friends here might see what they could do with the problem, and consider whether, with the aid of T-irons and concrete, they could construct or reconstruct this wonderful roof of the Romans.

Of course, the Romans, in using concrete, reinforced it with fire bricks or quarry tiles, as we should call them—large tiles either two feet or a foot square and 1½ in. or 2 in. thick; and with these they counteracted the shrinkage or other deficiencies of concrete, and formed temporary or permanent centerings which enabled them to get on well with the work.

The secret of the Roman concrete was a thing which, to him, was absolutely mysterious. He was very familiar with the great Roman fortress a mile and a half from Sandwich, known as Richborough Castle. This enclosure, covering several acres, was still surrounded by a great Roman wall. There was a wall there about 30 ft. high and 10 ft. thick and of great length, and when the railways were being constructed the barbarians destroyed the sea-wall to use the material for the purpose of the railway. On the return wall to this sea-front they excavated a great opening. It was more than high enough to walk into the cavity and it extended 8 ft. 6 in. in depth, and it left 4 in. beyond. The span of that opening was 50 ft.; therefore you have a concrete girder, say 20 ft. in depth and 50 ft. span and 8 ft. 6 in. in thickness without, of course, the slightest sign of crack or difficulty; and so it had been since the time, say forty or fifty years ago, when the cutting was made. So far as one could see, it was constructed of nothing but the materials on the spot—the gravel from the beach, flints, and the rough sort of half stone, inferior Kentish rag, which could be obtained on the spot, and the mortar appeared to be the ordinary chalk mortar.

Now what he wanted to know was, what did the Romans mix with that mortar which transformed it into a material as hard as our Portland cement; because inside that enormous wall, 10 ft. thick, the hardness of the material was just as good as it was on the outside? Of course, when the Romans left this country something or other which had been used in the mortar was no longer used, and the Normans, attempting to do buildings of the Roman character, made a fearful mess of it. Most of their central towers collapsed because of the mortar being bad; it became like sand, and the towers collapsed in consequence. Whether the Romans carried about with their armies Puzzolana, or volcanic ash, or something equivalent to that in its effects, he did not know; but it was a subject exceedingly well worth investigation.

Turning to another aspect of the question—the finish of this concrete—he happened to look at the Engineering Supplement of *The Times,* and he saw there an article about mica used in facing certain reinforced concrete telegraph poles. Well, of course, in granite and all materials like mica, and so on, you had a grand opportunity of lightening up the surface of concrete. Anyone who had tried, for instance, a rough cast of granite in Portland cement would see how very much better an effect could be obtained from the nature of the granite, particularly from the pink Leicestershire granite, than was available from the ordinary gravels or sands. And there were considerable possibilities by the use of acids in removing the excess of cement over certain portions, and in that way varying the monotony of the concrete surface.

There was no doubt that reinforced concrete was, in a way, a new material. As he understood the theory, reinforced concrete, and the practice of it, so far as it had gone, claimed that out of concrete and steel it made a new material which had qualities which were something more than the mere union of the two might imply. And, of course, there was the aspect of permanency about it which had not been given by any other steel or iron materials. The exact way in which it was going to be used and how eventually it would develop was a question which not one, two, or three generations were likely to solve, so far as the architectural truth of treatment is concerned.

Mr. E. P. Wells, J.P. — He objected to the statement that concrete *per se* was not beautiful; and he had been of the opinion all along that if we wanted to produce anything fit to look at there must be some kind of external treatment to produce a good effect. All kinds of means had been attempted by plastering and imitation stonework. Now and again one saw a building in concrete, especially in the country, that looked particularly well · if in town, where it got the usual London smoke, it then developed in a very short time the appearance of nothing but stucco.

Concrete could be made to look well if, when constructed, a surface work of mortar about 1½ in. or 2 in. in thickness were worked into it. That applied not only to the surface of the concrete itself, but also to any projections and mouldings there might be. And if that was done, even if no lines were shown to represent joints, it would then give a very respectable appearance. If the facing was made in plasters or cements or the limestones, it would in a very short time weather and become nearly white, and present almost the appearance of Portland stone. It took about two or three years to do the work, but it was also easy to tool, and at a very low expense a decent appearance could be given to a concrete surface *per se* without plaster work of any description being added to it.

At the present time, as references had been made to silos, there was no doubt about it that in buildings of this description, if you put them up purely as a concrete structure, without any relief, you might well stick people's backs up at their want of architectural beauty; but still, taking the size of the building and its height, with careful treatment, with a few projections and with pilasters, it could be made to look particularly well. As far as he could judge, from his experience, the thing was not to keep too many vertical lines. There should be a series of breaks in the silo, and then a very decent appearance indeed was obtained. But if there was a vertical basis right up to the top it became then intensely ugly.

There was no reason why in large structures, outside of buildings, especially in bridge work and where there were large arches, it should not be jointed in the usual method so as to represent a stone face, because if you take a vane arch without relief of any description you cannot say that it is beautiful unless you are some distance away from it. If there was a model to mount it on, then the illusion was all right; but otherwise it was not beautiful. There was no reason at all why in the treatment of concrete we should not adopt the usual architectural methods. It was an artificial stone, the same as a brick building was artificial. There was no reason why it should not be moulded and simply adapted and treated in a proper manner so that it was pleasing to the eye.

Professor Beresford Pite was of opinion that the engineer should never attempt architectural treatment in any of the works which he designed. He did not agree at all. He thought that now that reinforced concrete had come to the fore in the way that it had a great deal would rest with the engineer. He believed in collaboration between the engineer and the architect, but when they did have an engineer who had a moderate training also as an architect there was no reason why he should not combine both engineering and architecture: and it would be found that, especially in the case of very large buildings in the shape of factories, etc., you would get as good a design from the engineer as from the architect.

One point was raised by the last speaker with regard to the question of the mortars that were used by the Romans. He thought it was a well-known fact that in the old days the Romans simply used to slake their lime, generally for a period of two or three years before it was used, so that it became absolutely hydrated. There was no free lime in it whatever, and when it was mixed with the proper proportions of sand it produced a mortar that was almost equal to some Portland cement mortar of the present day.

Mr. E. Fiander Etchells, F.Phys.Soc. — Reading through the paper and listening to the discussion, it would appear that the problem to-day divided itself into two parts—first, whether we should consider reinforced concrete as a framing material having its own laws and its own proportions, and whether it should be exposed naked to the world: or whether we should only deal with it as a skeleton, and clothe it with some other material. Until the architects themselves had decided into what category reinforced concrete goes, he did not propose to express any opinion whatsoever.

Mr. Herbert Shepherd, A.R.I.B.A. — With regard to the question of joints, he thought it would be perfectly admissible, from a constructional point of view and from an artistic point of view, to use joints in reinforced concrete, because it must necessarily be that a huge building could not be done all in a day; however big the slab might be, you could not do it in a day. It seemed to him that it would be a very good thing to form the joint as the work stopped at a particular day.

Mr. Lucien Serraillier. — In the older methods of construction, such as masonry and brickwork, buildings constructed in that material relied entirely for their stability upon the weight of the material itself. With reinforced concrete the characteristic feature was tension, and the typical construction in reinforced concrete was, in his opinion, the cantilever; he thought an improvement would take the direction of large spans, cantilevers, and our ideas of proportion must necessarily be somewhat modified by this new form of construction.

THE LEC URER'S REPLY.

Professor Beresford Pite, in replying to the discussion, said that his paper was quite clear in stating that "the only method by which definite progress in an architecture of concrete will be possible to us is by the scholarly and critical employment of the traditional plastic forms of architecture." Now Mr. Scott seemed to have been under the impression that that was not the lecturer's opinion, but it was; and he thought that would also answer some of Mr. Wells's interesting remarks. One of the speakers had supported the idea that an engineer who gave attention to architecture could deal with the subject, and that it was possible to imagine certain principles of proportion and apply them to the new material irrespective of architectural examples in other materials. He would say flatly, clearly and plainly it was not, and there was an end of it—simply it was not possible. It was not possible to imagine a system of proportions for a new material apart from those ideas of proportion which were derived from others.

If they departed from the accepted proportions which were derived from traditional architecture they were at sea—completely at sea. They might talk about their surface; they might imagine that an occasional joint every other day or every other week, when there was no frost, would help them; but it would not. If they began to study architecture systematically, they would find that the sweet little exercises with which they employed themselves on such huge works were just little exercises in ignorance.

Might he again suggest that the point of this subject was that the aesthetic or the architectural treatment of concrete buildings wanted serious study, and it could only be undertaken by seriously laying hold of the principles of architecture, and that a sort of sporting shot at it—a sort of relying upon the average engineer's acquaintance with architecture who did not wish to make a fool of himself—was not enough. They might take it for gospel, if they pleased, there were rules and laws easily deducible with regard to the principles of architectural beauty. Now apply those principles to concrete; apply the principles of architectural tradition to concrete; the material itself would act through its texture upon those materials and create itself—would mark it for its own. They were perfectly at liberty to say they saw no harm in marking reinforced concrete in a cantilever bridge with the joints that represent a stone arch construction; but it was not architecture, and it was not engineering, and it was not an æsthetic treatment of concrete architecture.

THE CONCRETE INSTITUTE.

DISCUSSION of the Report on

THE STANDARDISATION OF DRAWINGS OF REINFORCED CONCRETE WORK.

At the summer meeting of the Concrete Institute, held in June, 1911, the report of the Reinforced Concrete Practice Standing Committee on "The Standardisation of Drawings of Reinforced Concrete Work" was submitted to the members. The discussion of the report was postponed to the Winter Session, and took place on January 11th last, Mr. F. E. Wentworth-Sheilds, M.Inst.C.E., Vice-President of the Institute, in the chair.

We would refer our readers to our July, 1911, issue, page 519, Vol. VI., for a copy of this report. The discussion was as follows:—

Mr. R. W. Vawdrey, A.M.Inst.C.E., Hon. Sec. of the Reinforced Concrete Practice Standing Committee, after reading the report of his Committee on "The Standardisation of Drawings of Reinforced Concrete Work," said it was clearly desirable that as much uniformity as possible should be attained in drawings; but he supposed all rules were only meant to be broken, and it was clearly quite impossible for any large drawing office to insist on absolute adherence to any such rules as had been suggested. The object of the report was to encourage the same style and methods rather than insist upon any details or attempt to recommend such insistence. His own personal opinion was that anything in the nature of colouring on drawings for reinforced concrete work, which were necessarily more or less intricate, should be avoided. He personally would like to lay more stress on that point than was made in the report generally. The same personal opinion applied to the method of marking to show concrete. So far from making matters clearer, it made them very much the reverse to attempt to show concrete by dots or hatching anything of that sort, especially with a view to reproduction of drawings. He thought everything should be either in black and white, or solidly blacked in.

The system of symbols was a good one, but in this the whole use of a system of symbols such as the Committee suggested was that they should be perfectly intelligible to everybody. That, he was afraid, would not be the case if they were used at the present moment, but in any drawing office in which a particular system of symbols, such as that suggested, could be used, the advantage in saving of space on the drawings and saving of time was very marked.

The same thing applied to standard lettering. It was quite impossible to hope that any standard lettering could be universally adopted, but if in any particular office that could be gradually brought about—the process would have to be gradual—it certainly would be a very great advantage.

Mr. L. Serraillier proposed a vote of thanks to Mr. Vawdrey for the trouble he had taken in the work of the Committee. He believed there was a method of making sections by rubbing the tracing paper over the back of a book where the cover was rough, and by rubbing the pencil over it they got some marks of a section. This they could do very quickly, and with a blue print they got the effect of hatching. Mr. Vawdrey had said the lettering should be as large as possible. Of course, the larger it was the more tendency there was that the appearance would not be good unless stencil plates were used. He very much favoured the use of stencils in drawings, and not block printing. The larger the block printing was the less regular it was and the uglier it appeared on the drawing.

Mr. S. Bylander seconded the vote of thanks, and in particular thanked Mr. Vawdrey for the very valuable remarks he had added to the reports.

Personally, he was most interested in the report as regards standardisation of notation and lettering. During his experience in America he found that the Americans had gained enormous advantage over the rest of the world, due to the fact that they had standardised their method of producing engineering work. The first thing that struck him most decidedly was the drafting room; they had standardised their method in great detail. Every office of good standing had a list of standard tables. These were used throughout the office by each member, and this enabled uniformity in design and methods.

Further, the title of a drawing was always placed on the lower right-hand corner, which was found very convenient in case of reference, when one had, say, forty or fifty drawings in a drawer, as one could thus easily find the particular drawing required.

Another thing which he believed was not quite realised was that the Americans used practically $\frac{1}{4}$ in. scale plans, instead of $\frac{1}{8}$ in., as was used here. They, therefore, used 1 in. scale details, instead of $\frac{1}{2}$ in. as here. He believed that this had been introduced from the fact that more constructional detail had to be introduced on the drawings, and, therefore, greater accuracy on the drawings was required, and in consequence a greater scale was employed. He wanted to mention this in connection with the report, as it appeared from that that the $\frac{1}{8}$ in. scale would be best. Personally, he believed that for steel work, or for plans for steel work, or where reinforced concrete was used, the $\frac{1}{4}$ in. scale was a better one, provided they did not get unreasonable sizes of drawings.

With regard to the $1\frac{1}{2}$ in. scale for details, he preferred the 1 in. scale, because the architectural details in this country were usually made to $\frac{1}{2}$ in., and the simplest method of transferring from one scale to another was by doubling the size. Further, he thought the 3 in. scale could be used wherever the 1 in. scale would not be sufficiently large.

With regard to the list of sizes of drawings, he believed it would be advisable to add something with regard to the size of order lists. That was a kind of drawing which would, he thought, be very much used in the future, much more so than hitherto. A very convenient size would be a quarter of the smaller size mentioned in the report—namely, 15 in. by 10 in. They, therefore, would establish a standard of drawings of different sizes obtained by halving the next larger size.

Mr. E. P. Wells: With reference to the report of the Reinforced Concrete Practice Standing Committee on "The Standardisation of Drawings," it was practically a unanimous one. Of course, there were certain variations that one would have liked to see, but it was an impossibility to satisfy the views of all. Taking it generally, the report was one which might be adopted in this country. Personally, he did not like to see all drawings necessarily the same, as it caused individuality to disappear to a very large extent.

With regard to scales, taking the general type of drawing that one had to use, the $\frac{1}{8}$ in. was very good for plans so long as there were no complications whatever. If they had plain, straightforward work, then they could get on to the $\frac{1}{8}$th scale plan everything that was necessary in the shape of reinforcing, and there was sufficient room also to state what the beams were composed of; but if it was a case of varying span both of floors and beams, then he often found that the $\frac{1}{8}$th was too small, and had invariably adopted $\frac{1}{4}$ in.

Regarding elevations for walls, $\frac{1}{4}$ in. was sufficiently large, practically, to show everything, unless there happened to be any architectural features requiring to be incorporated into the reinforced work, when the $\frac{1}{2}$ in. became too small. For beams and columns, as a rule, the $\frac{1}{2}$ in. was ample, because all the reinforcing could be shown in thick lines, which when reproduced came out very boldly, especially in the ferro-prussiate prints.

For sections of beams and columns he preferred the $1\frac{1}{2}$ in. scale. He had been in the habit of using it in all steelwork design for many years, very often multiples of $\frac{3}{8}$—$\frac{3}{4}$ and $1\frac{1}{2}$ in. At any rate, it was sufficiently large to show everything, and as a rule he thought it preferable to the 1 in.

Occasionally he found it necessary to use full size where there was great complication in rods, and where practically grout had sometimes to be used.

As to the size of the sheets, he liked to adhere to the Imperial size, but he never, if possible, got outside the Double Elephant. If he had to do so, then he got the Double Elephant by the Antiquarian length.

A plan which he had adopted, and which came in very handy at times, was to take some of the general work which was existing and put that in in pencil, with the reinforced work in ink. The result was exactly the same as working in two colours, in red and black, and it distinguished the new work from the old, and no mistake could possibly be made in the matter.

With regard to sections, and the question which had arisen about showing the concrete, he preferred it in all work, because once it was done there was an end of it. It was not as in the old days, when one had to trace everything.

Mr. P. M. Fraser, A.R.I.B.A., had not much to say with regard to the standardisation of drawings, except that he agreed with the previous speaker that the $\frac{1}{4}$ in. and the 1 in. were the two most useful scales that could possibly be employed. He was speaking as an architect, and Mr. Vawdrey did not make it quite clear to what type of drawing he was referring. There were two real types of drawing in connection with reinforced concrete building, the architectural and the reinforced concrete drawing, and to a certain extent they were separate. He used the $\frac{1}{4}$ in. scale generally. It was said that drawings of that size were too large; but they could be cut in half or in quarter, and there was not the slightest objection to cut them up to any size one liked. It was a great convenience to have drawings cut up, and if they had any question as to any part of the building they could take a small drawing which was easily handled, about 2 ft. square, to that particular part, and need not walk about the building with drawings under their arms.

Mr. E Fiander Etchells, F.Phys.Soc., suggested that it would be desirable that the prints or drawings should be indelible black-line photo prints on white linen, which would enable any alterations or additions to be readily shown. An objection to the blue prints was the danger to eyesight in examining them day after day and hour after hour in a dark, foggy place like London. With regard to hatching on blue prints, which were difficult to trace, they could be laid over a thick sheet of plate glass with the ordinary electric light behind, and they would then show up very plainly and could be traced readily. He also suggested that tracings should also be made on the unglazed side of the linen, the reason being, again, that any pencil notes could be readily made, which was difficult on the glazed side.

It would also be desirable that the north point should always be indicated on all plans, and also all plans of the same building should have the north point leading in the same direction as far as practicable.

All sections should bear the same title or reference, clearly indicating the position of the section plane and the direction of the point of view, etc.; and also it would be desirable that all views should be projected from one another, as far as possible, and be placed in the geometrically correct planes.

Some confusion was occasioned by those who learned to draw before they had studied geometry, putting the left elevation on to the left side. Another desirable feature in reinforced concrete drawings would be that the proportion and nature of the concrete and the minimum crushing load at one month or at three months should be clearly indicated on the plans.

Mr. R. W. Vawdrey: Everybody agreed that white prints were very much pleasanter to deal with, and better for the eyes. The only object in using blue prints was that they were very cheap. Alterations could be made on blue prints. Assuming that cost was no object, white prints were infinitely better.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

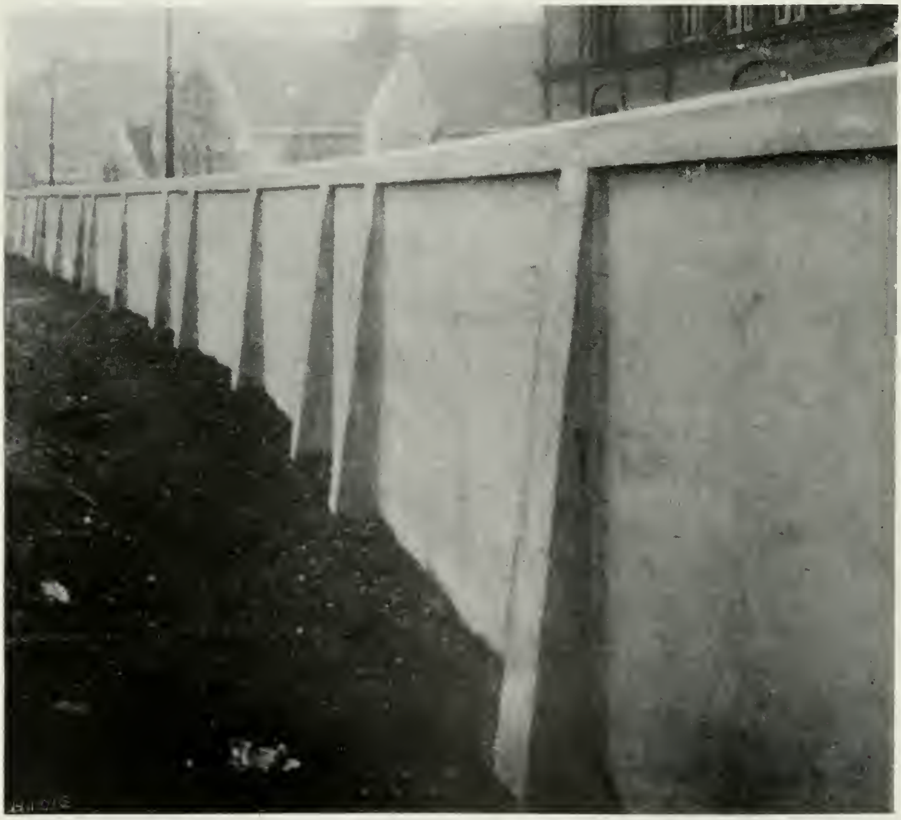
Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE BOUNDARY WALL AT WEST HARTLEPOOL, FOR THE NORTH-EASTERN RAILWAY COMPANY.

An interesting example of reinforced concrete boundary wall construction is shown in the illustrations given herewith of the Expanded Steel-concrete boundary wall lately completed at West Hartlepool, to the designs and under the supervision of Mr. W. D. Rudgard, North Eastern Railway District Engineer.

The wall varies in total height from 7 ft. 6 in. to 9 ft. 6 in., and the line drawing on page 224 shows the plan and elevation at the higher portion.

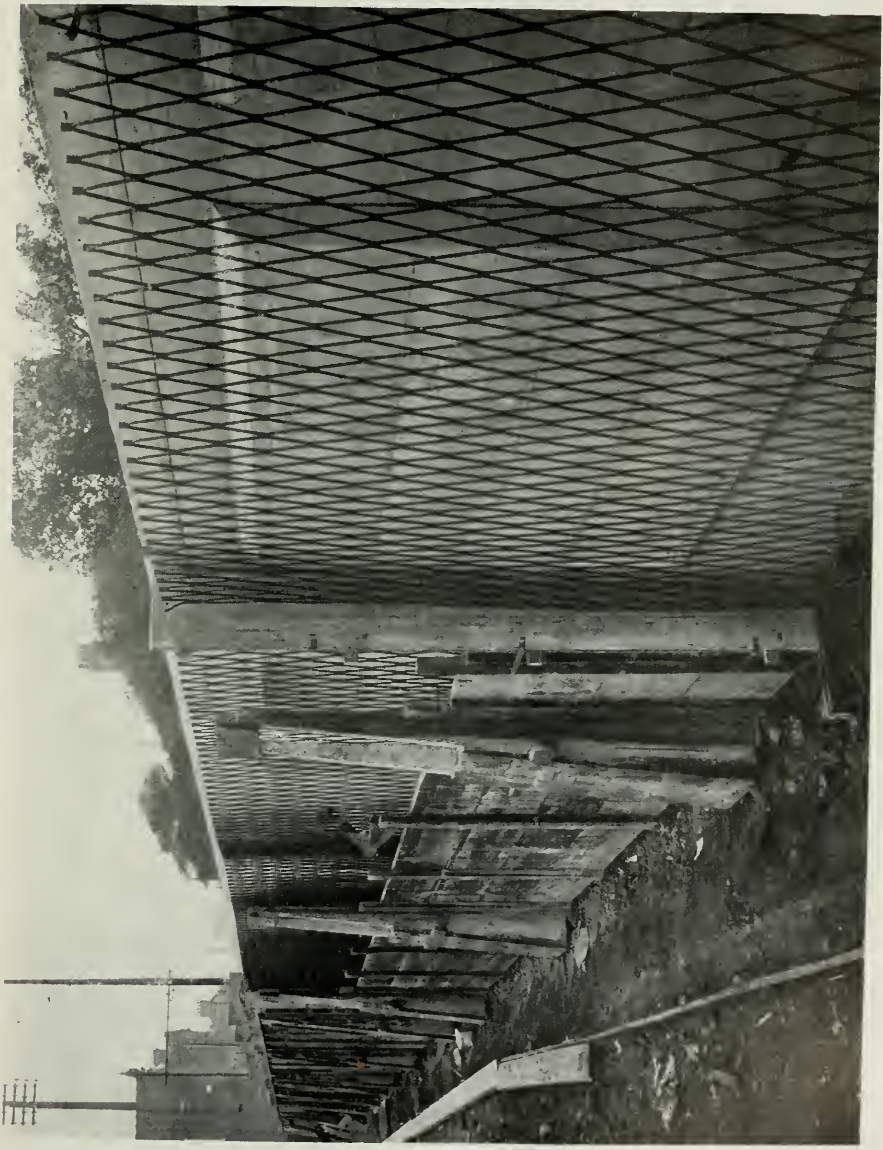
The wall is 6½ in. in thickness, reduced to 4½ in. in the panels, with footings as shown in the drawing. The road face is panelled and the back face is buttressed. The raking buttresses are 1 ft. in width and about 13 ft. centre to centre, while the panels are 5 ft. long and 4 ft. high, with pilasters 1 ft. wide.



View showing the buttressed face of the wall to the railway.
Reinforced Concrete Boundary Wall at West Hartlepool.

Breaks were made at about 32 ft. centres in the walling, and ¾-in. wood strips inserted, to allow for expansion and contraction.

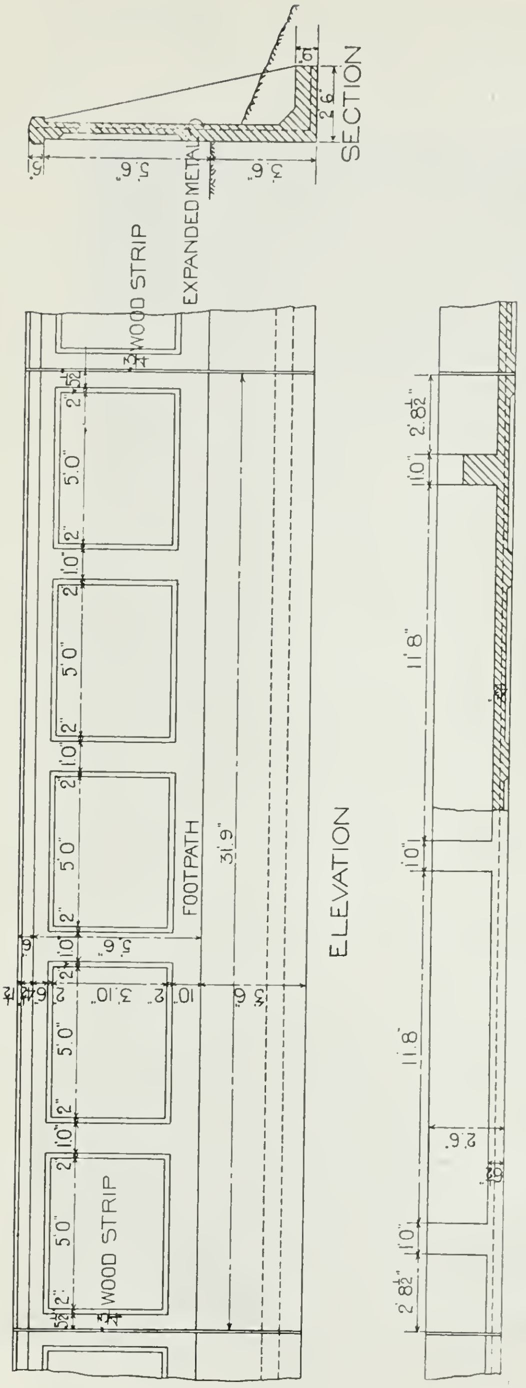
The photographs show the work in course of construction and completed, and from them it will be seen that the wall is pleasing to the eye as well as efficient.



View showing Expanded Steel Reinforcement in position.

Reinforced Concrete Boundary Wall at West Hartlepool.

The illustration on this page shows the temporary timbering with the Expanded Steel reinforcement in position ready to receive the concrete. It will be noticed that while the timbering for one side was erected complete before the placing of the concrete, that on the other side was erected a few boards at a time, to allow of the concrete being placed in courses and thoroughly rammed and consolidated in position.



Plan and Elevation

Reinforced Concrete Boundary Wall at West Hartlepool.

The other views show respectively the buttressed face to the railway and the panelled face to the roadway.

From the photographs can be seen the ease with which Expanded Steel reinforcement may be handled, and the fact that the work was done by direct labour proves that there is no necessity for specially skilled labour in connection with it.

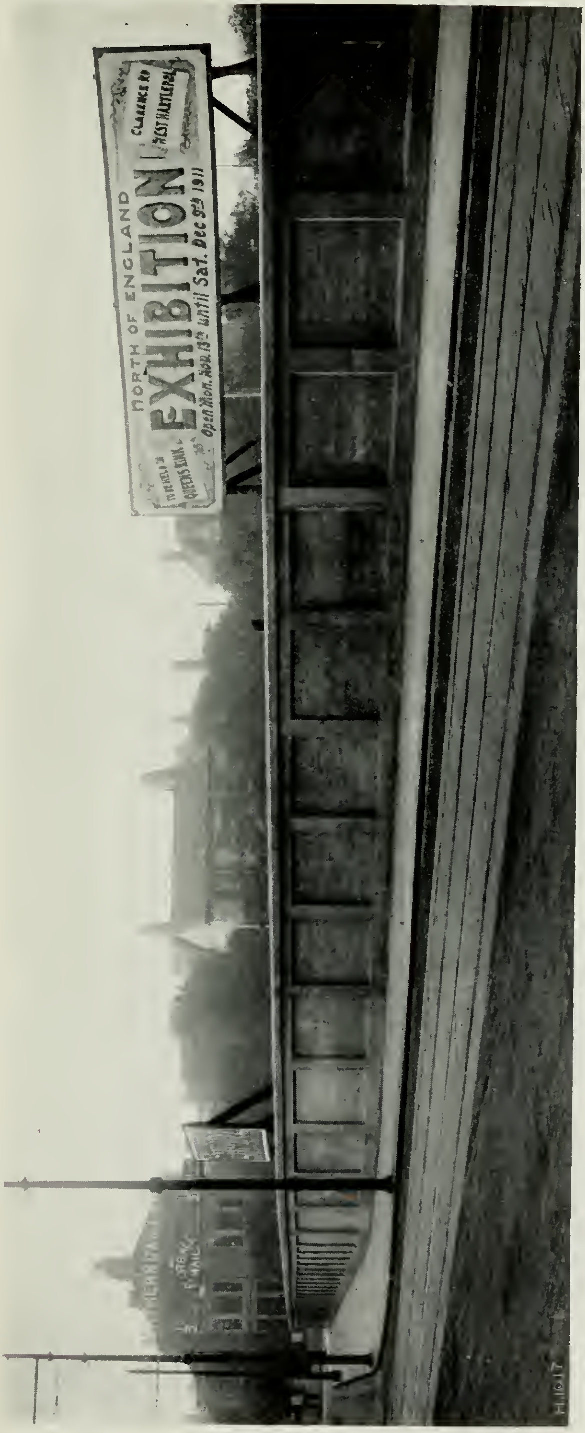
The Expanded Steel reinforcement was supplied by The Expanded Metal Co., Ltd., of London and West Hartlepool.

REINFORCED CONCRETE CONSTRUCTION IN A STORE AND LOFT, LOS ANGELES, CAL.

In the building of which we show illustrations on pages 226 and 227, and which is now under construction at Los Angeles for F. W. Braun, wholesale chemical supplier, concrete has been selected as the most desirable for many reasons.

It is built on what may be appropriately called a cantilever system. The floors are supported on columns dividing the floor area in 18-ft. panels; the columns are supported on reinforced footings which are almost flat. The top of these footings, however, is cast in an inverted hopper, 8 ft. × 8 ft. square and 18 in. deep. These inverted hoppers are turned upright and used again at the neck of each column just under the floor slab.

In this system the base of the column is made in the ordinary way with circular wrapped reinforcement and rods started from dowels secured in top of footing, while the head of the column has reinforcements projecting through the column from either side, making cantilever arms in six directions, and the floor slab is


NORTH OF ENGLAND
TO BE HELD IN QUEENS RINK
EXHIBITION
Open Mon. Nov. 13th until Sat. Dec 9th 1911
CLARENCE RD WEST HARTLEPOOL
H.1617

View showing panelled face of the wall.

REINFORCED CONCRETE BOUNDARY WALL AT WEST HARTLEPOOL.

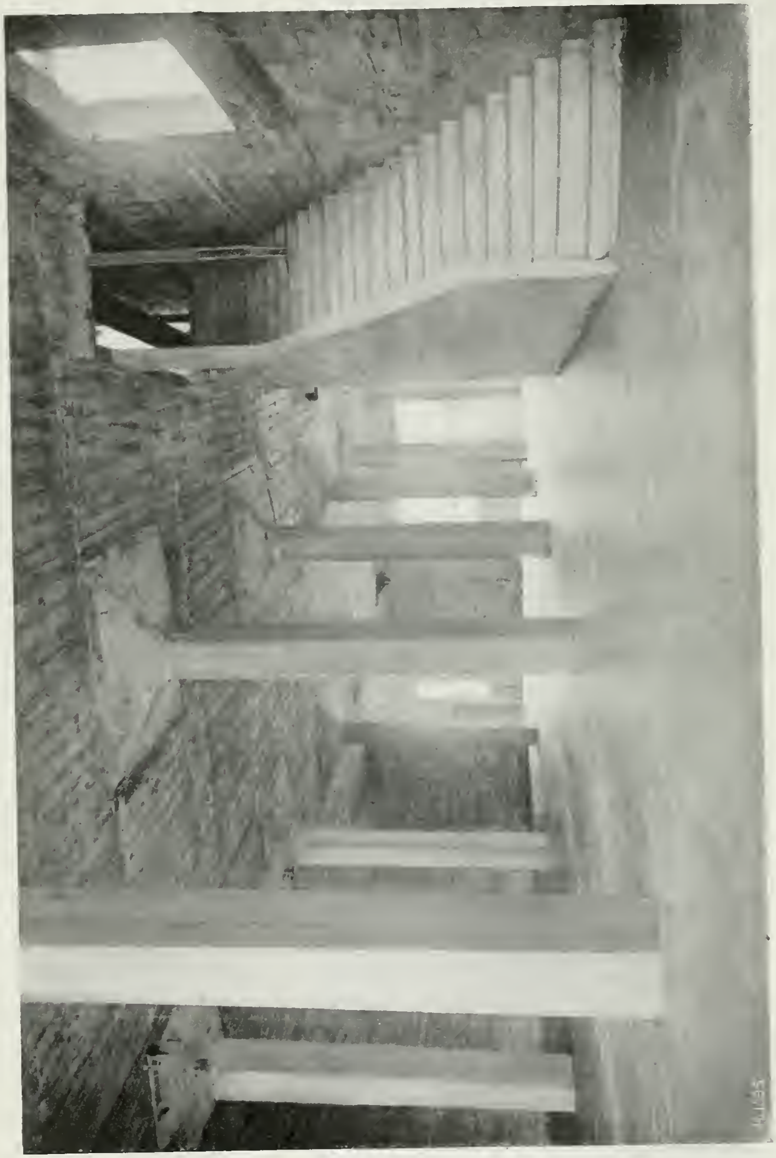
supported on these cantilever brackets. Over the cantilever arms the floor slab reinforcement extends in all directions, the head of the columns being made wide enough to receive it. The floors may thus be said to have beams 8 ft. wide, since the slab steel is the strongest as it passes at right angles over the head of the columns, but no extra depth is given to this portion of the floor.

There is a continuous batter along the walls which reduces the span of floor slab, extending from columns to walls. Cantilever rods in the column heads are formed with 20 rods $\frac{3}{4}$ in. diameter, and the steel in the columns is of 1-in. round rod.

The columns have the rods set in a wrought-iron band in a circular manner. The circular column steel is wrapped with number 3 wire, spiral wrapping, the spacing for the wires being 2 in.; the basement columns have 20 rods 1 in. in diameter, the first and second storey columns have 16 rods 1 in. in diameter. These rods are joined near the floor line in each case by using pipe sleeves in the ordinary manner. Reinforcement for floor slabs is of $\frac{3}{8}$-in. twisted steel, rising at right angles to column heads, and American steel wire mesh of number 3 wire, spaced 4 in., running diagonally across column heads.

The walls are 12 in. thick for all side and rear walls, and 16 in. for front walls. The reinforcements for walls are $\frac{3}{8}$-in. rods, forming rectangles 18 in. square.

Column rods 1 in. in diameter are placed in the wall, making reinforced piers directly opposite each row of columns supporting the floor slabs. Stairways are all of reinforced concrete and are formed on slabs 5 in. thick and the treads and risers project above the slab; reinforcements in the stair slab are of $\frac{3}{8}$-in. twisted rods 6 in. apart in each direction; the same applies to each floor. The balustrade is 4 in. thick and $2\frac{1}{2}$ ft. in height and reinforcement extends into floor slab. The reinforcement in the building equals about 400 tons.



Interior of Building, showing finished column heads.
Reinforced Concrete Store and Loft, Los Angeles, California

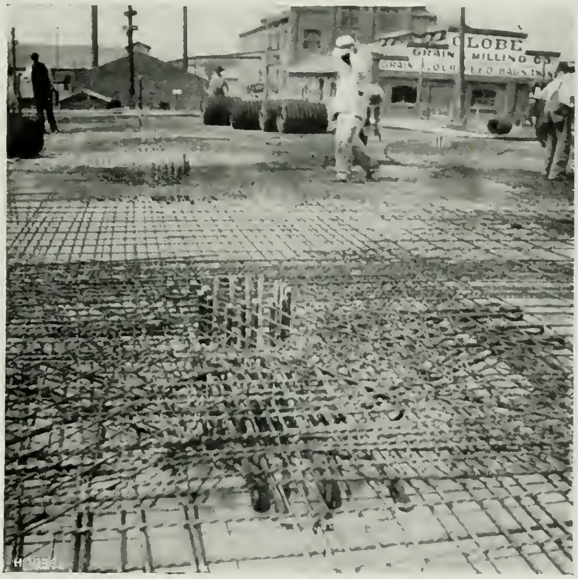
The front of the building was marked into blocks by using bevelled strips $\frac{1}{2}$ in. thick by $1\frac{1}{2}$ in. wide, these strips being nailed to forms dividing the blocks into rectangles 14 in. × 21 in. each. When the forms for front of building were stripped the walls were cleaned and brushed with a thick coating of neat cement, applied with a stiff hair brush.

Cement required on the building totalled about 5,000 barrels.

The building is designed for five stories, but is now being erected to the extent of three stories and a roof slab of the fourth is being built, which, while acting as a roof, will be available as a future floor. The columns in the basement are 22 in. in diameter and on the other floors 20 in. in diameter.

The height of the stories varies, that of the basement being 10 ft. floor to ceiling, first floor 16 ft., second 11 ft., and the third 14 ft. high, according to the purpose for which they are required. This lack of uniformity, however, lengthened the time of construction, which was 90 days to complete.

The roof slab, which later will be the fourth floor, is to be covered with one layer of asphaltum roof felt and one layer of heavy waterproof paper, which is considered strong enough to form a decking, allowing the roof slab to be walked on and used for the purpose of mixing chemicals.



View showing floor reinforcement.
Reinforced Concrete Store and Loft, Los Angeles, Cal.

The concrete floors have been thoroughly tested, with a result that they showed a test of 500 lb. per sq. ft., which gave a deflection of $\frac{1}{8}$ in. only in centre of 18-ft. panel.

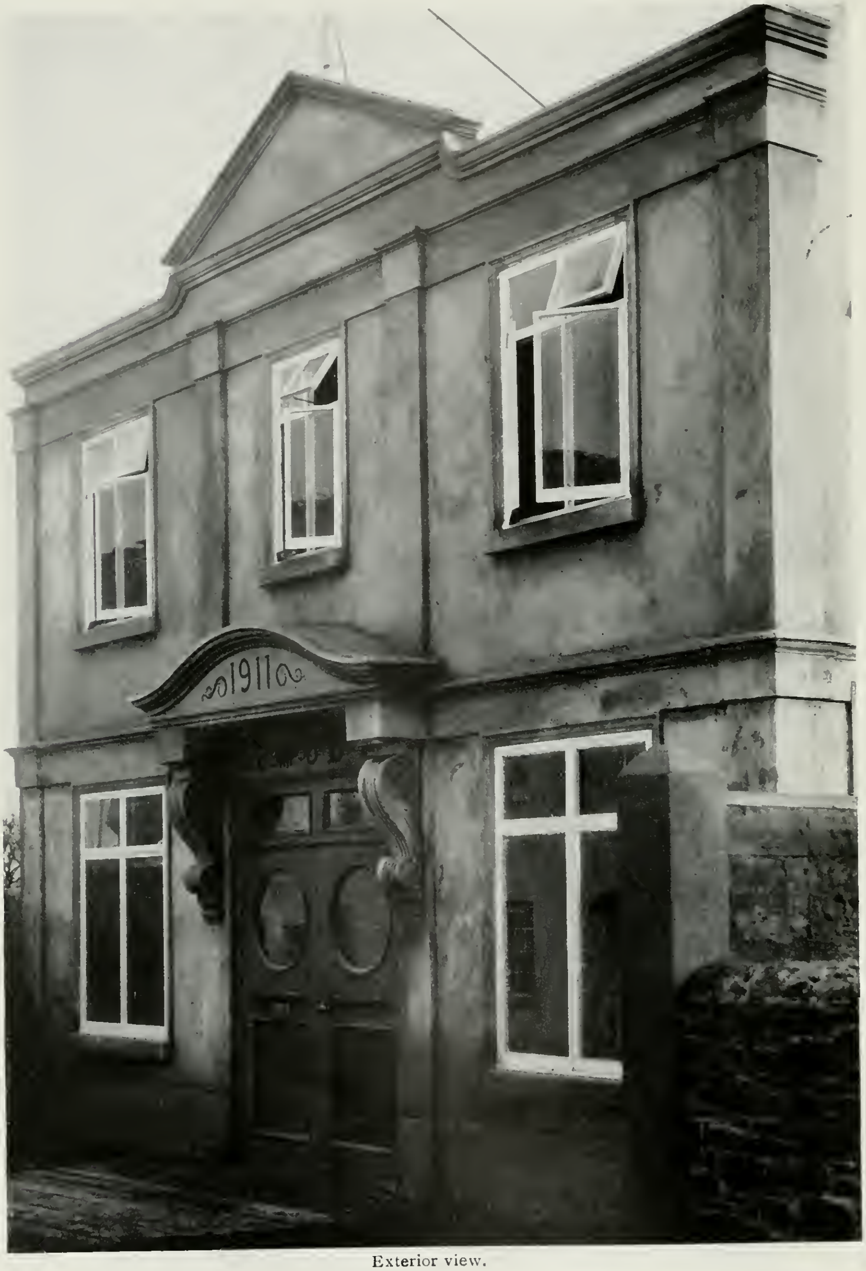
The total value of the job is calculated at $50,000 (£10,000).

DRILL HALL, SOUTH MOLTON.

The building has been erected to provide adequate quarters for the local Company, and is situated close to the centre of the town, fronting on a main road.

The erection consists of a two-storeyed block at each end, front and back, the actual drill hall being of one storey only, and placed between the double storey ends.

The new hall, which is situate in North Street, is a convenient and well-built structure of reinforced concrete with fireproof partition slabs. The entrance hall measures 17 ft. 6 in. by 7 ft. 6 in. On the right is an orderly room 17 ft. 6 in. by 10 ft. 6 in., and on the left is an officers' room of the same dimensions. The main hall is a fine room 16 ft. high, measuring 30 ft. by 60 ft., a view of which is shown in our illustration on page 229. This will be used for drill purposes. Beyond this there is an armoury 26 ft. by 14 ft. 6 in. Upstairs is a lecture hall 17 ft. 6 in. by 15 ft. 6 in., and a room for the instructor. The instructor's living quarters are over the main entrance.



Exterior view.

Reinforced Concrete Drill Hall, South Molton.

The entire structure is composed of reinforced concrete, the whole of the weights being transmitted to pillars, the spaces between being filled in with thin 4-in. concrete walls, having reinforcement lacing into the pillars and thus becoming monolithic in construction.

The pillars are 12 in. × 12 in. and 14 in. × 14 in., and the beams vary from 12 in. × 8 in. to 30 in. × 12 in.

The first floor and roofs are also of reinforced concrete, averaging 5 in. in thickness, the front and back blocks having flat roofs.

The main hall is spanned with a sweeped roof also in concrete, the circular concrete ribs being designed to act as straight beams, thus exerting practically no thrust on the side pillars.

The whole of these roofs are covered with green Ruberoid, the concrete walls being rendered over externally in cement.

Interior of Drill Hall.
Reinforced Concrete Drill Hall, South Molton.

The internal partitions are formed with 2½-in. Mack partition slabs, and the whole of the interior is finished off in ordinary lime plastering.

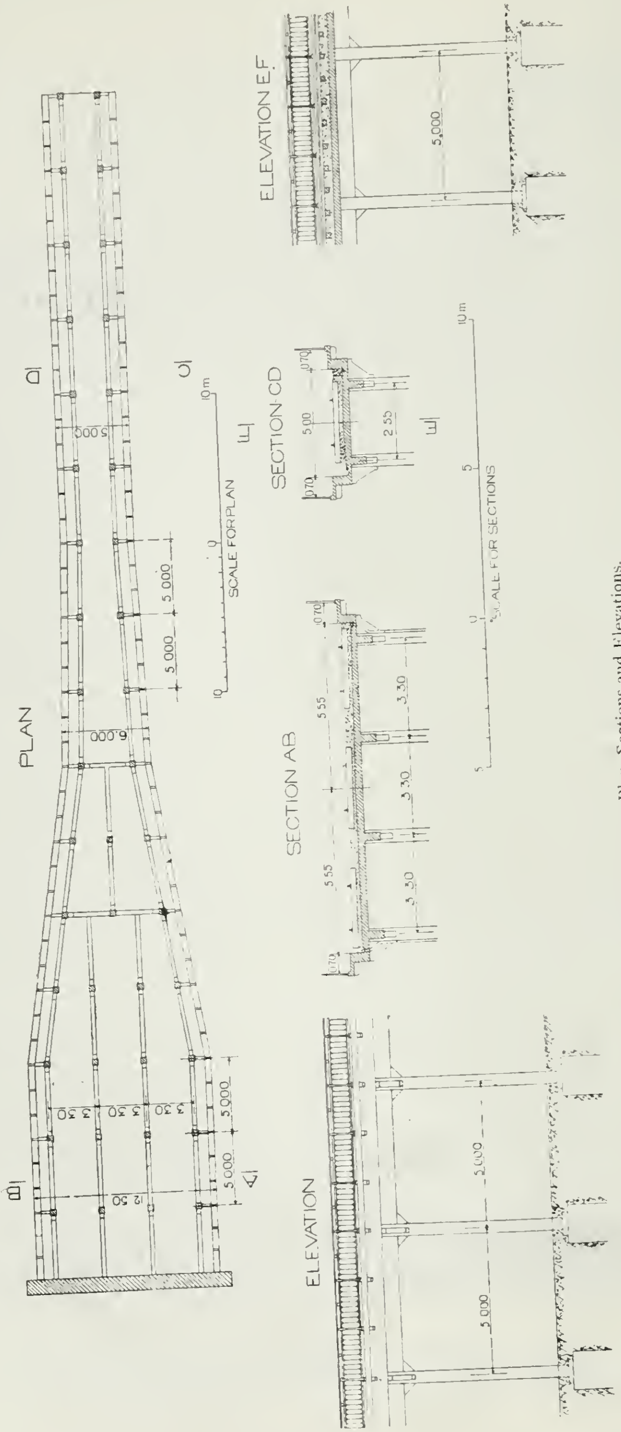
The contractors for the general building were Messrs. Woolaway & Sons, Barnstaple; the contractors for the plumbing and gas-fitting were Messrs. H. R. Williams & Co., Barnstaple, Mr. Spencer Edwards, P.A.S.I., of Barnstaple, being the architect.

REINFORCED CONCRETE RAILWAY VIADUCT, GENNEVILLIERS GAS WORKS, NEAR PARIS.

The works of the Gennevilliers Gas Co. were opened in 1906, and provide gas for lighting three communes with a total population of over a million. The works cover an area of 143 acres, and are located on the banks of the Seine, but are cut in two by a public thoroughfare. This thoroughfare is crossed by the Gas Co.'s light railway, and in connection with this railway two reinforced concrete viaducts were erected by the Company.

The viaducts are of the same dimensions, the second one being a duplicate of the first in every respect. The total length of each is about 700 ft.

At the widest portion of the work the viaduct carries four railway lines of standard gauge, and calculated for locomotives of 50 tons and wagons of 30 tons fully loaded.

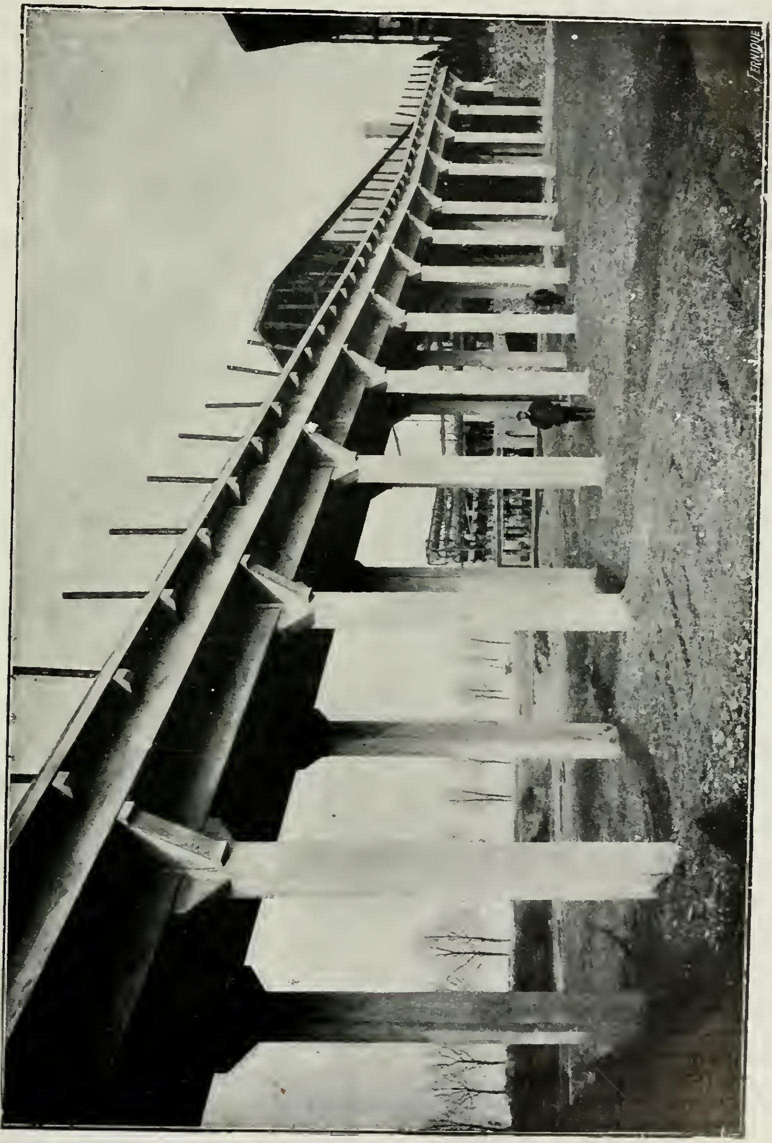


Plan, Sections and Elevations.

Reinforced Concrete Railway Viaduct at Gennevilliers Gas Works, near Paris

The work was carried out under the supervision of M. Stinville, Chief Engineer of the Gas Works, M. Edmond Coignet, of Paris, being the contractor.

Reinforced concrete was chosen specially for this work on account of its resistance to sulphurous gases, and the economy realised by carrying out the work in that material over similar stone construction amounted, we understand, to about 30 per cent.



Reinforced Concrete Railway Viaduct at Gennevilliers Gas Works, near Paris.

As far as possible the whole of the extensive buildings in connection with these gas works have been erected in reinforced concrete. The works produce a considerable quantity of highly inflammable by-products, and the more bulky of these are stored in large reinforced concrete tanks, each capable of holding 500 tons.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"Reinforced Concrete—Mechanics and Elementary Design." By John P. Brooks.

New York: McGraw-Hill Book Co., 239 West 39th Street. 220 pp. + x.

Contents.—Historical Sketch—The Component Parts—Cement—Sand—The Aggregate—Tests of Reinforced Concrete—Analysis of Stresses—Beam Theories—T Beams—Columns of Concrete—Beams Supported in More than Two Places—Elementary Design—Economical Proportions of Beams—Report of the Joint Committee—Design of a T Beam—Design of Reinforced Concrete Retaining Walls—Design of a Cantilever Retaining Wall—Design of a Counterfort Retaining Wall—The Arch-Algebraic Analysis—The Arch-Graphic Analysis—Design of a Reinforced Concrete Elastic Arch Ring—Design of a Beam and Girder Floor—Design of a Reinforced Concrete Hollow Dam.

This book should be very interesting to engineers in this country owing to the fact that it is written by an expert in a country where the use of reinforced concrete has been far more extensive than has been the case on this side of the ocean, and interesting comparisons may be drawn between the different methods. The author himself states that corrugated bars are used far more in America than in Europe—hooked ends being more generally used in the latter; and, while advocating the deformed bar, he has also a good word to say for the hook in certain cases. A really interesting section of the volume is that devoted to the analysis of stresses, which is illustrated with various plates and diagrams and interspersed with problems, and although the English student may find it somewhat strange to study from a volume written to suit American students, he will certainly obtain a great deal of useful knowledge from this part of the work. Retaining walls are dealt with in detail, and far more notes are devoted to these and the design of arches than is the case with elementary books in this country. The general instructions given for each example in the elementary design are interesting, as they show the student the sequence of the work and afford a good guide as to the method of procedure, even the size of the drawing sheet and scale being given. The volume is well written and illustrated, and, although we should not like to see it supersede any of the English text-books, it should certainly form a useful addition to the library of technical books and prove of much service to the reader.

"The Theory of Structures," by Charles M. Spofford, S.B.

New York: McGraw-Hill Book Company, 239 West 39th Street. 411 pp. + xiii.

Contents.—Outer and Inner Forces—Law of Statics, Reactions, Shears and Moments—Influence Lines—Concentrated Load Systems—Beam Design—Plate Girder Design—Simple Trusses—Bridge Trusses with Secondary Web Systems—Trusses with Multiple Web Systems, Lateral and Portal Bracing, Transverse Bents, Viaduct Towers—Cantilever Bridges—Three-hinged Arches—Design of Columns and Tension Members—Pin and Riveted Truss Joints—Graphical Statics—Deflection and Camber—Continuous and Partially Continuous Girders and Swing Bridge Reactions.

This is a volume dealing with the complete theory of structural engineering, and it contains a great deal of information which, generally speaking, is presented in a concise and clear manner. A large proportion of the book is given up to those problems which are only met with in railway engineering, and, apart from their theoretical interest, may be passed over by the designer of ordinary buildings. We are sorry to see that no table of notation is given, as this we consider is indispensable in a book of this character, and more especially if it is likely to be read by structural engineering students in this country. Beams fixed at both ends are also not dealt with, and this is to be regretted, bearing in mind that the theory is of great importance at the present time, especially as reinforced concrete allows this condition to be fulfilled, and the book treats so fully of other matters.

Influence lines are much used, and although of more value when live loads have to be dealt with than in any other case, their use is interesting to a student, and should enable him to fully grasp the behaviour of various structures under loads situated at different definite points. A curve of shears, or moments, is a curve the ordinate to which at any point shows the shear or moment at that point caused by a set of loads, fixed in magnitude and position. The ordinate to the influence line shows instead the shear or moment at the section for which the influence line is drawn due to a load of unity acting at the point where the ordinate is measured. The chapter on deflections is a good one and treats very fully of this subject. Generally speaking, we consider the volume a very good one, and it should prove a very useful book of reference for those engaged in structural engineering.

"Spons' Architects' & Builders' Price Book and Diary."

Published by E. & F. Spon Ltd., Haymarket, S.W. Price 5/- net the two volumes.

Spons' "Architects' and Builders' Price Book and Diary" for 1912, which is again published in two handy volumes, as in 1911, is one of the most valuable reference books for architects, surveyors, and all connected in any way with the building trade. It is fully up to the high standard of former issues, and the prices, etc., have been brought thoroughly up to date.

The memoranda section is carefully cross-referenced, and there are a great many valuable notes on various matters of interest to architects and builders.

"Builders' Quantities" by H. M. Lewis.

Published by E. & F. N. Spon Ltd., 57 Haymarket, S.W. Price 1/6.

This is an elementary work on the subject of builders' quantities, and meets a want for a cheap handbook within reach of every building student and others in the building trade.

It contains a great amount of information as to the customary methods of measurement in the "London Standard." It gives examples of how to measure up in every trade, as well as methods of taking off, abstracting and billing. We think the work will prove useful to those engaged in the various trades connected with building.

"Ornamental Cement Work," by Oliver Wheatly.

Published by Scott, Greenwood & Son, 8 Broadway, Ludgate, E.C. Price 5/- net.

Contents.—Introduction — Materials and Workshops — Mould Making and Templates — Technique — Choice of Ornaments—Extended Uses.

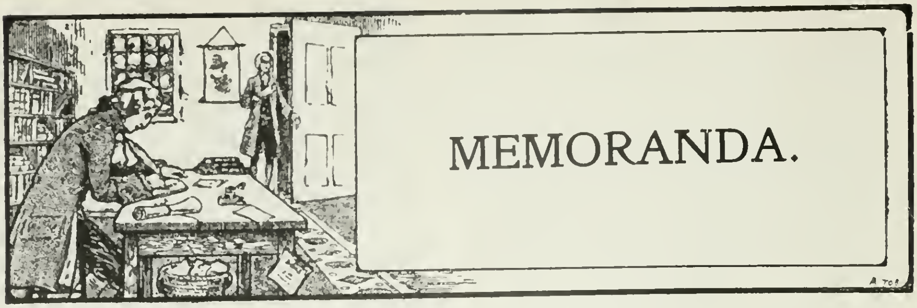
This book, as its name implies, deals only with the application of Portland cement in ornamental work, and is intended as a handbook of the principal facts concerning such application. It has been published to place before the artist-craftsman particulars of a material which lends itself easily to reproduction of a permanent character, and one which needs a very small outlay. Owing to its plastic nature cement can be applied in very numerous ways to ornamental work, and particulars and illustrations of these are given in this handbook. There is a chapter dealing with the various moulds and templates necessary, and another gives some very artistic designs suitable for cement moulding.

Altogether, this little book is very well got up, and should be found useful by anyone wishing to carry out ornamental work in Portland cement.

"Lectures on Cement," by Bertram Blount F.I.C.

Published by The Institute of Chemistry, 30 Bloomsbury Square, London, W.C. Price 2/6 net.

The Institute of Chemistry recently inaugurated a scheme under which Fellows who have special knowledge and experience in various branches of work are engaged to deliver lectures chiefly for the benefit of young chemists and advanced students. Among the first of these were two lectures on "Cement," delivered by Mr. Bertram Blount, F.I.C. From the synopsis of these, which appeared in our December, 1911, and January, 1912, numbers, the interesting and useful nature of the lectures will be gathered. They should certainly be studied by all those connected in any way with cement, for they indicate the scope and character of work actually carried out, as distinct from purely academic training.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

New Offices for the Metropolitan Railway.—The Metropolitan Railway Company has recently placed in competition a certain number of engineering firms specialising in the design of reinforced concrete structures, for the new offices which the Company intend to erect on the site of the Baker Street Station.

Messrs. Edmond Coignet, Ltd., of 20 Victoria Street, Westminster, have been selected for the preparation of the working drawings, and the work is to be executed on their system under the control of Mr. W. Willox, M.I.C.E., Chief Engineer of the Metropolitan Railway Company.

The principal dimensions of this large building, which is to be constructed entirely in reinforced concrete, are as follows:—The total length of the front elevation will be 140 ft., and the height, measured from the foundations to the roof, will be approximately 90 ft. The back portion of the structure will have two wings, measuring respectively 111 ft. in length by 38 ft. in width, and 100 ft. by 43 ft. for the smaller wing, the latter being connected to an existing building by a footbridge 82 ft. in span, 12 ft. wide, and with a height of 15 ft.

A retaining wall in reinforced concrete of a length of 152 ft. will run parallel to the front elevation. The total length of the retaining wall, including the returns, will be 212 ft., the height being 24 ft.

The walls of the building are to be entirely in reinforced concrete of a thickness of 6 in., and for the principal elevation the wall is to be faced with brickwork.

The building will be composed of a lower basement, basement, booking hall floor, ground floor, first, second, and third floors, and a flat roof. The superloads up to the first floor included, are to be 150 lb. per sq. ft., the second and third being 90 lb. per sq. ft., and the roof 40 lb. per sq. ft.

The total superficial area of floors and roof in reinforced concrete will be approximately 65,000 sq. ft.

The Coignet system, which has been adopted for this work, is at present also being used for the construction of the new bridge at King's Cross Station for the same Company.

The Junior Institution of Engineers.—A handy sized booklet has just been issued by this Institution, dealing with "Its Origin and Aims," and makes interesting reading.

In the paragraph dealing with the inception of the Institution, it is interesting to note that the Society came into being at the works of the late firm of Messrs. Maudsley, Sons & Field, London, in 1884. Amongst the officers, Presidents and Vice-Presidents, the names of many famous men can be noticed in the list that is given. Mention is made also of Honorary Members, one of whom generally gives a lecture each year, many of the discourses thus delivered having merited and obtained world-wide renown.

The rooms are kept open on Friday evenings during the months of October to May inclusive, in order that the weekly periodicals can be perused and social intercourse promoted. In connection with each Friday evening, the opportunity is taken for the discussion of various subjects in a very informal sort of way.

The subscription for membership is £1 1s. for town and 16s. for country members. A copy of the booklet may be obtained free on application to the Secretary, 39 Victoria Street, Westminster, London, S.W.

The Society of Engineers' Status Prize.—The Council of the Society of Engineers (Incorporated) may award in 1912 two premiums of books or instruments to the value of £8 8s. and £4 4s., as first and second prizes, respectively, for approved essays on the subject of "How to Improve the Status of Engineers and Engineering, with Special Reference to Consulting Engineers." The Council reserve the right to withhold either or both of the premiums if the essays received are not of the required standard of merit. The competition is open to all, but application for detailed particulars should be made to the Secretary before entering. The last date for receiving essays is Friday, May 31st, 1912.

Mica Surfacing for Concrete Poles.—Ground mica is being used extensively for surfacing concrete work, according to *The Times* Engineering Supplement. It has proved very effective for this purpose, as it gives an artistic finish to the work and adds life and sparkle to the surface, taking away the flat dead appearance that is common to concrete. About 5 lb. of mica is sufficient to cover 100 sq. ft. The electric light columns in Lincoln Park, Chicago, Illinois, were treated in this way, crushed red granite being used so that the finished surface had the appearance of polished granite. The granite and mica surfacing material was applied to the inner surface of the square iron trough in which that part of the columns extending above the ground was cast. When the trough was filled, the top or lid was screwed down, pressing the concrete into all the lines and corners of the mould. After the cement had set perfectly, part of the mould was removed to permit the post to cure more rapidly. The post was not removed from the mould until it had set for 24 hours at least. After it was perfectly dry the surface was scrubbed with muriatic acid to remove the cement on the outer face of the granite and mica and leave a clean surface closely resembling that of granite.

Austrian Tests of Portland Cements.— The annual statistics of the results of the Government tests conducted under the care of Professor A. Hanisch, and published in the *Mitteilungen des Technologischen Gewerbemuseums* have just been issued for the year 1910. The author points to the steady increase in the tensile strength of the samples coming under his notice. At 28 days the average values rose from 375·4 lb. per sq. in. to 403·8 lb. per sq. in., and in the same way the strength under compression in 28 days was 5,944 lb. per sq. in., as contrasted with 5,360 lb. per sq. in. in the previous year. The proportion of tensile to compressive strength was in the mean 1 to 14, and it would seem from this fact that the figure so frequently quoted for this ratio of 1 to 10 needs to be revised. The number of samples which failed in various ways in the tests to ascertain constancy of volume was somewhat remarkable, though on the whole fairly good results under this head were obtained. The absolute setting time in the bulk of the speciments examined ranged between four and eight hours, which seems excessive. The specific gravity of the raw cement was in the majority of the cements between 2·95 and 3·10; the same samples heated to redness gave results varying from 3·10 to 3·20. On the sieve of 5,806 meshes to the sq. in. the residue in the majority of cases was 0 to 2 per cent., and on the sieve with 31,613 meshes the residue varied from 15 to 25 per cent. All the tests were conducted in accordance with the conditions laid down by the Austrian Engineers and Architects' Society.—*The Times.*

Reinforced Concrete Armour Plates for Battleships.—Reinforced concrete armour plates for battleships are declared practical by Signor F. d'Adda, a European engineer. He proposes to protect battleships with reinforced concrete plates, 5-in. thick, the concrete being so proportioned and deposited as to bring the strongest part of it to the exterior surface. This engineer claims that such plates would offer as great a resistance to projectiles as a 1-in. steel plate, and the specific gravity of it would only be 2.25 against 8.1 for the steel plate.—*Concrete Age.*

Self-anchored Piles.—A comparatively new American practice is to drive concrete and reinforced concrete piles in swamp or sandy ground so as to obtain bulbous ends. The method pursued is to drive down two steel tubes, one fitting in the other. When driven to a sufficient depth the inner tube is withdrawn, the base being collapsible and controlled by steel rods, thus the core is removed. Concrete is then poured in and rammed tight, until sufficient resistance is felt. When desired, a steel rod or spiral wire skeleton may be sunk in the hole, but the main object is to force the concrete to expand below the outer steel tube, which it does in the form of a bulb or mushroom, producing perfect anchoring. The outer steel tube is then withdrawn. In river and marine work, where piles are often grouped close together, a monolithic base or foundation may be obtained in this way, even if each pile is some feet apart from any other.

The Sinking of Reinforced Concrete Piers.—Having to sink a number of reinforced concrete piers of 4 ft. diameter, with the base diameter of 6 ft. 6 in., an American contracting firm has effected considerable economy by employing steel cylinders, which were driven by a special pile-driver and utilised as cofferdams, wherein timber moulds were set up for the concrete work. After having fulfilled this purpose in the construction of one reinforced concrete pier, each cylinder was drawn up and driven in another place for similar use. The steel cylinders were 50 ft. long by 7 ft. 7 in. in diameter, and weighed 17 tons each. The monkey on the pile-driver weighed 5,000 lb. and the force of the blow delivered was distributed over the cylinder head by a timber dolly of massive construction. The pull necessary for withdrawal of the cylinder was approximately 50 tons.—*The Engineer.*

The Examination of Foreign Matters Added to Cement.—The following is an abstract of an article by M. J. Cartiaux in the *Revue des Materiaux de Construction et des Travaux Publics*:—

The tendency of the price of cement to become lower in course of time makes it all the more necessary that simple methods should be available for the detection of adulteration. At present the detection of these additions is often neglected because of the assumed difficulty in testing for them. The object of the present article is to suggest simple methods by means of which some of these additions may be detected without the necessity of employing delicate and costly apparatus.

The Adulterants Used.—The chief additions made to cement are crushed slags—both granulated and slowly cooled—hydraulic lime, barium sulphate, sand, plaster of Paris, ground bricks, and limestone. These various substances may be detected as follows:

The Use of a Lens.—The best results are obtained by examining the residue on a sieve with 4,900 holes per sq. cm. by means of a lens magnifying eight diameters. The lens may be so mounted that its field is exactly one sq. cm. in area, and a good idea of the proportion of any easily observable constituents may thus be obtained. The addition of granulated slag is readily seen, the "matite" being easily recognised. The presence of characteristic grains of yellow or black with a vitreous appearance is another important indication of slag. Care must be taken not to confuse these black grains with those of carbon; the latter may be removed by heating the cement so as to burn the carbon, and re-examining the calcined product. Particles of metallic iron may be detected with the aid of a magnet.

Specific Gravity.—Most adulterants of cement have a different specific gravity and may be detected by this means. All fresh Portland cements which have a specific gravity below 3·05 must be regarded with suspicion, and as probably containing slag, chalk, etc.

Although the following procedure is not always correct, it is an invaluable preliminary test: (1) Ascertain the specific gravity of the residue on a sieve with 4,900 holes per sq. cm.; (2) ascertain the specific gravity of the material which passes through such a sieve. With an unadulterated cement the two figures will be almost identical, and will certainly not differ by more than 0·015, but with an adulterated cement the difference will be much more marked. A cement to which 10 per cent. of slag has been added will show a specific gravity in the fine portion which is 0·04 or more higher than that of the coarse portion. This addition of dense material may, however, be masked by the use of chalk or other light substances. If the latter are used alone they will reduce the specific gravity of the fine powder to below that of the coarse. In making this test very large pieces should be avoided, as they are apt to destroy the accuracy of the result.

Chemical Analysis.—This should be made on the residue and on the fine powder separately, as more valuable results are thus obtained. Thus an excess of silica in the residue implies the presence of slag.

Another important characteristic, which is too often disregarded, is the proportion of manganese in the coarse and fine portions respectively. Slags usually contain up to 3 per cent. of this element, whilst pure cement is quite free from more than traces of it. The following volumetric method may be used: Two grammes of the sample to be examined is treated with 20 c.c. hydrochloric acid (conc.) and 5 c.c. nitric acid and is evaporated to dryness, taken up with 20 c.c. more hydrochloric acid, and neutralised with 10 grammes zinc oxide and 20 grammes zinc sulphate. The mixture is warmed to a temperature of 86 to 90° C. and titrated with a standard solution of potassium permanganate corresponding to 0·0017 grammes iron per c.c. After the evaporation has been effected (and this needs little or no attention) the test does not take ten minutes, so that its use is highly convenient without being too tiresome to be used regularly. If this test yields a negative result, it does not prove that no slag has been added, but if the proportion of manganese found is considerable it may be taken as a positive proof that added slag is present. Whilst too much reliance must not be placed on the actual percentage of manganese, the addition of 10 per cent. of slag will increase the manganese from nil to about 0·6 per cent. The addition of hydraulic chalk is shown by the presence of a notable proportion of free chalk in both the fine and coarse powder.

Methylene Iodide.—Le Chatelier has proposed to separate the materials lighter than cement by means of a liquid of intermediate density. For this purpose he chose methylene iodide diluted with benzol to a density of 2·93. One gramme of the sample is placed in a tiny flask of about 10 c.c. capacity and the flask is filled with the fluid just mentioned. The vessel is then given a brisk rotary movement whereby the lighter materials float to the upper part of the fluid and their volume is measured on the neck of the flask. The test is made more easily if the residue on a 4,900 sieve is used. If it is desired to separate the materials a separating funnel is used instead of a flask, and the lower liquid drawn off below through a tap. They may be separated from the methyl iodide by washing with water or water and ether, and may then be examined by any of the methods previously named.

Skipton Extensions.—Extensive additions are to be made to the English Sewing Cotton Company's Mills at Skipton from the designs and under the supervision of Mr. J. Hartley, architect of Skipton, consisting of a new warehouse, mechanic shops, etc., at the Belle Vue Mills. It is proposed to use reinforced concrete very extensively in these extensions, and the contract has been let to the Trussed Concrete Steel Co., Ltd.

Cocoa Works Extensions.—Messrs. Rowntree and Co., of York, have just completed further new extensions to their Cocoa Works at York and erected a handsome building for the special benefit of their workpeople, including recreation and dining halls along with other administrative offices, wherein reinforced concrete has been largely used, designed, and carried out by their own architect, Mr. J. W. Swaine.

County Council New Buildings.—The West Riding of Yorkshire County Council have just let large contracts for the erection of new headquarters for the West Riding Constabulary at Wakefield from the designs of their county architect, Mr. J. Vickers Edwards, wherein the main floors are to be carried out in reinforced concrete.

New Baths at Nelson.—The Nelson Corporation are about to erect new swimming baths from the designs of their borough engineer, Mr. W. Shackleton. It is intended to carry these out very largely in reinforced concrete, and tenders have recently been asked for this work.

Concrete Groyne at Clacton-on-Sea.—For the further protection of the west beach at Clacton-on-Sea it has been decided to construct a big concrete groyne near the jetty, at an estimated cost of £2,250.

Reinforced Concrete in the Reconstruction of a Fish Quay in Dundee.—The Montrose Harbour Trustees have approved unanimously of a scheme submitted by Mr. J. Hannay Thompson, Dundee Harbour Engineer, for the reconstruction of the fish quay with reinforced concrete and timber fenders at an estimated cost of £4,700.

Cleaning Concrete Moulds.—A simple method of cleaning out moulds employed in reinforced concrete work is to blow steam on them from a hose pipe. It is stated by an American engineer that steam is more efficient and more economical than any other medium for the purpose.

Concrete Telegraph Poles and Pillar Boxes.—Wooden and iron telegraph poles are being supplanted in New Zealand by poles made of reinforced concrete.

It is said that tests made with an 18-ft. pole showed that it could carry a greater

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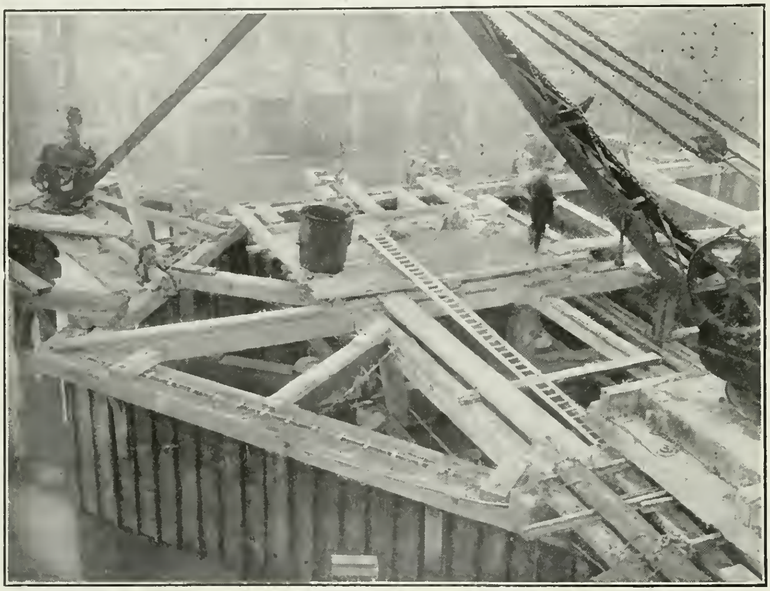

Illustration shows one of three cofferdams constructed with our steel piles at L.C.C. Reconstruction, Deptford Creek Bridge.

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weight than either wood or iron. The average life of a wooden or iron pole is about nine years, while it is considered that the reinforced concrete poles are practically everlasting.

CORRESPONDENCE.

Concrete Beams with Double Reinforcement.

Sir,—Glancing at *Fig.* 3 in Mr. Andrew's article, it struck me as peculiar that at the point where the tension in concrete ceases to be considered the moment of resistance of a concrete beam should suddenly increase. Checking, therefore, the construction of his diagram, I find the following miscalculations, which render the diagram useless :—*First Stage.*—Modulus of Section $= \frac{I}{\frac{9d}{16}} = \cdot 4014bd^2$ instead of $\cdot 3613bd^2$. This increases the bending moment to about $96 \cdot 34bd^2$.

Second Stage.—The solution of the quadratic equation gives $n = \cdot 423d$, not $\cdot 435d$, and the bending moment comes to about $212 \cdot 76bd^2$.

Yours faithfully, D. W. R.

The Editor, *Concrete and Constructional Engineering.*

ERRATUM.

In our description of the Maverick Cotton Mills, Boston, on pages 148 and 149 of our February issue, through an oversight the illustrations inserted were those of the Massachusetts Cotton Warehouse at Lowell, Mass., constructed by the Aberthaw Construction Company, of Boston. Both these buildings were designed by Messrs. Lockwood, Greene & Co., of Boston.

TRADE NOTICES.

The Piketty System.— Messrs. Paul Piketty & Co. have opened offices at 14 to 18 Bloomsbury Street, London, W.C., for the exploitation of their system of reinforced concrete in the United Kingdom. They are equipped to render expert services to architects, engineers and surveyors upon all matters appertaining to reinforced concrete. Tenders for the construction of "Piketty" reinforced concrete structures are submitted by their British licensed contractors.

We are informed that the "Piketty" system is somewhat similar to the Hennebique system, differing only in the disposition and form of the members that resist shearing stresses. Messrs. Piketty & Co. use round steel for stirrups, which are locked to the upper horizontal reinforcement.

The system was introduced some twelve years ago by Mr. Paul Piketty, civil engineer and public works contractor, of Paris, in quite a modest way; to-day, it is one of the largest reinforced concrete systems used in Europe.

Great care is exercised in the calculation and preparation of designs and working drawings. The extensive technical staff is cleverly departmentalised, so as to produce its work efficiently and with dispatch. Messrs. Piketty make a point of only acting through architects, engineers and surveyors, and do not undertake commissions direct with building owners or local authorities. This certainly seems a fair and sensible business course to adopt. As a private firm they become responsible for their work to their licensed contractors, who in turn are responsible to the building owner and his technical adviser.

We understand that the firm have in the press a large handbook, fully describing and illustrating the Piketty system, together with photographs of various types of works carried out. We hope to review it in our next issue. The firm have recently published a British Specification for the use of architects, engineers and surveyors, describing the manner in which their system is constructed. Application for copies should be made to Messrs. Paul Piketty & Co. at their London offices.

Mr. R. H. Neal, A.M.Inst.C.E., M.I.M.E., who has had considerable practical experience of reinforced concrete in the United Kingdom, is associated with Mr. Piketty, and has taken over the general management of the British business, which is quite distinct from the Parisian house.

Kennedy Bar-bending Machine.—We have received a catalogue recently issued dealing with the Kennedy Bar-bending Machine. This machine, which is practically the only one of its kind made in this country, should be of interest to all workers in reinforced concrete for the bending of either round or square bars up to $1\frac{1}{2}$ in. diameter.

Recent orders for this machine have been received from the Admiralty and the Commissioners for the Union of South Africa, together with such well-known contractors as Messrs. William Moss and Sons, Ltd., Leslie and Co., Ltd., Perry and Co. (Bow), Ltd., Trollope and Colls.

We have been asked to point out particularly that the inventor of this machine would be willing to fix up contracts for bending on the job. All interested who visit the Building Exhibition at Rusholme, Manchester, to be held from the 9th to the 23rd of March, should make a point of seeing a demonstration at Stand No. 23, Row D.

Further particulars can be obtained from Mr. W. Kennedy, inventor and patentee, of 11 Furzeham Road, West Drayton, Middlesex.

The Indented Bar and Concrete Engineering Company's New Directors.—We have been informed that Mr. L. Serraillier, manager, and Mr. R. W. Vawdrey, chief engineer, of the Indented Bar and Concrete Engineering Co., Ltd., have been appointed directors of that Company.

CONTRACTS.

The Armoured Tubular Flooring Co., Ltd., of 53 Victoria Street, S.W., have recently secured the following contracts for their flooring:

Kent County Council Offices, Maidstone. Architect, Mr. F. W. Ruck, County Architect.
Messrs. David Rowen's premises, Glasgow. Engineers, Messrs. W. Bridges & Co.
Eagle Brewery, Oxford. Architect, Mr. W. J. Wilkins, Oxford.
Drill Hall, Canterbury. Architect, Mr. F. G. Hayward, Dover.
Royal Naval Hospital, Chatham, for the Admiralty.
Post Office for H.M. Office of Works, Barrow-in-Furness.
Manchester University Extension. Architect, Mr. Paul Waterhouse.
Royal Beach Hotel, Southsea. Architects, Sir A. Blomfield & Sons
Convent at Boscombe. Architects, Messrs. Carew & Finn.

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Porous Pipes
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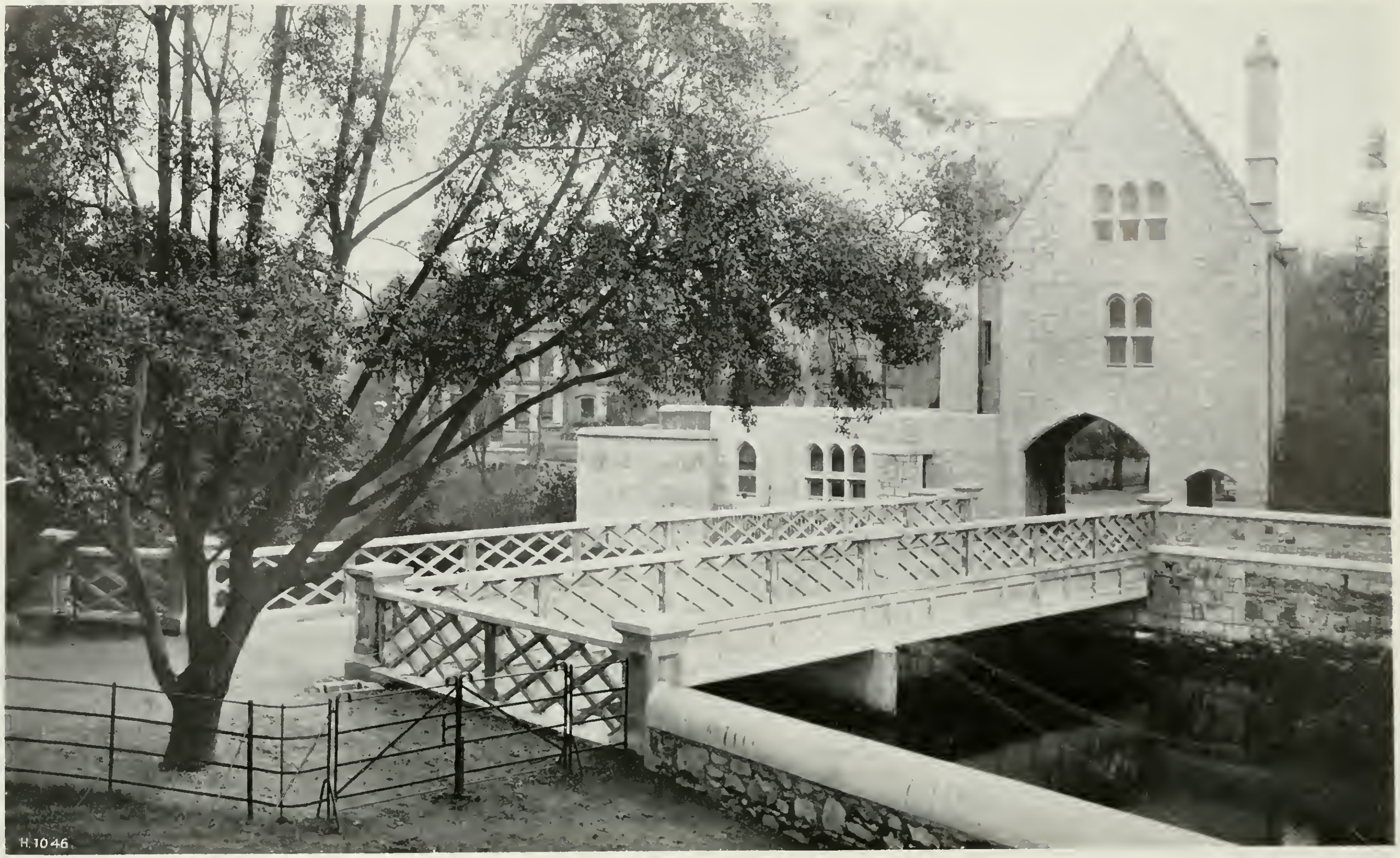


(*For description see page* 306)

Reinforced Concrete Bridge in the Grounds of University College, Cork.

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume VII. No. 4. LONDON, APRIL, 1912.

EDITORIAL NOTES.

LONDON COUNTY COUNCIL REGULATIONS FOR REINFORCED CONCRETE.

WE have recently published in our columns the regulations which are proposed by the London County Council with respect to the construction of buildings wholly or partly of reinforced concrete, power to draft which was given in the London County Council General Powers Act, 1909.

In general, these regulations have our whole-hearted approval, and we think they have been extremely well drafted, and record safe practice. We would point out that these regulations do not apply to any and every kind of reinforced concrete work to be put into buildings in London, and the whole of the various authorities, such as gas companies, dock companies, etc., are exempt in the same way as they are from the operation of the London Building Acts. Moreover, these regulations apply only to what may be termed frame buildings in reinforced concrete, in which the walls are reduced from the thickness provided under the former London Building Acts.

Fortunately these rules for reinforced concrete construction do not form part of a statute—*i.e.,* included in an Act of Parliament—but are made independently by the Superintending Architect under the provisions of an Act of Parliament. We wish that powers could be obtained by the London County Council upon some future occasion to rescind certain parts of the former Building Acts and enable the Superintending Architect to draft regulations, at least for all matters affecting the science and practice of constructing buildings. The advantage of these "Regulations," as distinct from "a statute," is that they can be altered from time to time as practice and science progress.

Coming to detailed consideration of the draft regulations, we find it provided in Part 2, Section 6, that the dead load of a building shall consist of the permanent construction comprised therein, and among such construction is included partitions. As, however, the partitions are seldom placed definitely upon the plans when the building is commenced, some alteration or other always taking place, it would be wiser to provide that partitions may be reckoned as temporary structures and included in the superimposed loads, seeing that it is permitted to reckon so-called temporary partitions, weighing quite as much as ordinary so-called permanent partitions, as included in the loads specified for various classes of buildings under Section 9. It would be well to have an allowance to include in such loads partitions of a stated weight, whether temporary or permanent, and if in the case of certain buildings the partitions were really permanent and able to be taken account of in the calculations, and the construction made such as to sustain them, then it would be logical to permit a reduction of the superimposed loads by such amount as should be stated to be included in the loads set forth. For instance, supposing it were permitted to put temporary or permanent partitions upon a floor without special calculations up to, say, 20 lb. per sq. ft. of the floor area, then if the partitions are separately provided for in the calculations of any beam or beams sustaining them, it should be permissible to reduce the superimposed load by the said 20 lb. We consider this would be the truly scientific way of regarding floor loads, for no doubt the 70 lb. which is provided for with ordinary floors in domestic buildings is quite excessive unless partitions be included therein; 50 lb. would be ample, and is allowed for tall buildings in America.

As regards Sections 21 to 23, it should be recollected in determining the depth of a beam in respect to the span and the limitation of the deflection of a beam, that a "beam," in the meaning of these regulations, may be a cantilever, and in the case of a cantilever sustaining a concentrated load at the end, the deflection is much more than in a "beam" in the ordinary sense of the word. As the deflection depends upon the load as well as the span we do not see quite where this regulation is thoroughly logical. The limitation of the deflection should be a matter for the engineer designing the job to consider, as it might be desirable in many circumstances to put in a much shallower beam than is here provided.

We wish that Sections 27A to 27I could be omitted, because from a theoretical point of view they are not at all adequate and will have little or no practical advantage. All conditions of loading have not been provided for, and the bending moments set forth are for a single concentrated load at the centre of a beam or at the free end of a cantilever, or for an evenly distributed load. Seeing that there are many other conditions of loading which are extremely frequent in practice, to which the sections do not apply, such cases then coming under Section 32, where it is provided that the beams or slabs are to be calculated by the accepted formulas of modern engineering practice, such a provision could quite well apply to all cases. Apparently the object in setting forth these rules for determining bending moments is to provide some allowances for imperfect fixity, varying moments of inertia, settlement of supports, and partial or irregular distribution of loads. This could have been much better met by inserting clauses reading somewhat as follows :—

(1) In the case of supposedly fixed beams the bending moment shall be determined on the assumptions of a uniform moment of inertia and supports maintained at assumed levels, and to such bending moments at any and every point there shall be added :

(*a*) an allowance of 5 per cent. where the moment of inertia is not uniform and where the effect of such variation be not determined;

(*b*) an allowance of 5 per cent. for unequal settlement of supports; and

(*c*) a still further allowance of 10 per cent. where absolute fixity is doubtful.

(2) In the case of continuous beams the bending moment shall be determined on the assumptions that continuity does not extend over more than three spans, and to such bending moments there shall be added :

(*a*) where the moment of inertia is not uniform and where the effect of such variation is not determined, an allowance of 5 per cent.;

(*b*) to provide for continuity over more than three spans where such continuity may exist, an allowance of 6 per cent. on the bending moments of the dead load and an allowance of 10 per cent. on the bending moments of the superimposed loads about the middle of intermediate spans, and an allowance of 5 per cent. on the bending moments of both the dead and superimposed loads about the adjoining supports, but no increase need be made about the middle of end spans;

(*c*) to provide for unequal settlement of supports a still further allowance of 5 per cent.

It might also be advantageous to state that for evenly distributed loading on beams of equal spans extending over more than three supports, the maximum bending moments could, where not determined by precise calculation, be taken at $+\frac{Wl}{12}$ about the middle of the intermediate spans and $-\frac{Wl}{12}$ at the intermediate supports, and $+\frac{Wl}{10}$ about the middles of end spans, and $-\frac{Wl}{12}$ at adjoining supports. It is thought that the above would be ample provision to make in practice.

Sections 29A, B and D require remodelling in accordance with the foregoing, and we may call attention to the fact that to Section 29B, if it be retained, words should be added to make it necessary to take the bending moment at the middle the same as is required for the ends under this section, while in Clause 29D we would point out that the bending moment given for the centre of the span of a rectangular slab does not accord with Clause 271. It would be better to omit the first terms of the formulæ—namely, the factors for bending moments—from this section, and simply let the second terms of the formulæ apply to a reduction of the load to be assumed as coming upon the shorter or the longer spans respectively. The previous rules for determining bending moments would then apply to these equivalent loads.

In Sections 33, 132A and 132B the working stresses upon concrete are regulated, but, in view of the fact that safe shear stresses will vary directly with the compressive resistance, it would be better to state the working shear stress as a percentage of the compressive resistance; Clause 132B would then operate also in respect to shear for other proportions than the two recorded in Clause 33. Further, as regards this question of shear stress, distinction should be made between the shear stress in a beam not reinforced against diagonal tension, and pure shear where the diagonal tension is taken not by the concrete. We are quite in agreement as to allowing 60 lb. for 1 :2 :4 concrete in the former case, but where all the tensile web stresses are provided for by the reinforcement, or where pure shearing stress occurs (*i.e.*, punching shear) as in a foundation, or in the case of a column standing on a slab or beam, and also along planes such as at the connection of the rib with the flange of a tee beam, and at the junction of the rib of a beam with the support, we are in favour of allowing the shear stress to be as much as 50 per cent. of the compressive stress permitted on the concrete. In justification of this we would refer to the experiments of Professor Spofford and Professor Talbot, and also to experiments referred to in Professor Mörsch's book, it being shown in the latter that steel cannot come into play in pure shear until the concrete is cracked, after which the concrete comes into bearing compression on the rods and in such case American experiments recorded on pp. 57 to 59 of our issue for January, 1910, have shown that a great deal more steel would be required than it would be practicable to use for such a purpose. Thus unless some provision be made for calculating concrete alone as sustaining the pure shear stress according to the value we suggest, we feel that the regulations would not be quite on the safe side, for, as we have said, the fact that the tension component of the shear may be calculated to be taken entirely by the steel is not sufficient, because if the steel can only come into play after the concrete has cracked there would be some settlement before the concrete came into bearing against the rods, which would mean somewhat large cracks that would be likely to result in corrosion of the steel.

In this same section it is required that the tensile stress in web reinforcement shall be taken at 12,000 lb. per sq. in., and that the shearing stress shall also be taken at the same figure. With the latter we agree, because it is then a question of pure shearing stress, but to our mind there is evident confusion in the former as to the true function of the web reinforcement. The provision in the London County Council regulations is evidently derived from the requirements set forth in the Royal Institute of British Architects' Joint Committee's second report, but we would point out that that report permits one to take the concrete, even in the presence of shear members, as capable of resisting 60 lb., the remainder of the shear being taken by the steel, and this procedure would also be permitted under Section 40 of these regulations. This might well be unsafe in the case of large tee beams where the shear is considerable, for, if most of the concrete in the effective section be cracked, as the Joint Committee and the London County Council assume can be the case, then that part cannot take shear. Thus practically the whole shear would have to be taken by the steel. Consequently, we can see how it is possible, if the concrete in the rib has been assumed to take 60 lb. shear all over the section, for the real stress coming upon the web reinforcement to be far in excess of 12,000 lb. per sq. in. It would be far safer to provide that where web reinforcement is required at all the concrete shall not be calculated to take any of the shear, but that the tensile stress in the web reinforcement shall be permitted to be of the same value as the direct tensile stress in longitudinal members, for it clearly has to serve the same function as the web members of lattice or N girders. This would emphasise the function for which the web members are provided--namely, to resist the tensile stresses in the web—the concrete serving in its turn to resist the compressive stresses, which it is quite able to do notwithstanding any cracks.

We are glad to see that the direct tensile stress on mild steel is put at 17,000 lb. instead of 16,000 lb. per sq. in., as in the Royal Institute of British Architects' Joint Committee's report, because if 7½ tons is the stress customarily permitted in ordinary structural steelwork where large sections are likely to be more uneven in quality than the small sections used in reinforced concrete, and are, furthermore, not adequately protected against corrosion as are the sections employed in reinforced concrete work, there is all the more reason for taking a higher stress than $7\frac{1}{2}$ tons rather than a lower stress, such as 16,000 lb. For our part, we think it would be quite right, in view of the quality of steel required by the regulations, to permit 18,000 lb.

As regards Section 47, it seems to us undesirable to include the word cantilever under the term "beam," unless it be stated that the word shall only apply to Sections 47 to 79, because if not, it clashes with Clause 27A and 27B, and also with Clause 23, to which latter we have referred above. We would prefer to see a cantilever called a cantilever and not a beam; it would only mean inserting the words "and cantilevers" after the word "beams" in Sections 19, 21 to 23, 57, 65, 66, 71 to 74, 116 and 150.

In Sections 43 to 46, while we agree that it is good enough to take the modular ratio as 15 in calculating beams in practice, we think that the modular ratio for pillars and compressive reinforcement generally should be based as a percentage of the compressive stress permitted on the concrete; Clause 46 would then need to have the words "in beams, cantilevers and slabs" inserted after the word "concrete," while another clause should be added that in calculating compressive reinforcement generally the modular ratio should be obtained by some such formulæ as the following: $m = 30 - {\cdot}025_c$, with an upper limit of 4.

Section 52 is perhaps the best provision that can be made for calculating the compressive reinforcement in beams, but it is probable that it will be proved to be unsatisfactory where much steel is provided in the top of a beam, for the rule might require the steel to be of greater area in the compression area of the beam than in the tensile area. In such case it appears that the concrete in compression might be ignored.

The distance of the anchors required by Section 54 for the compressive steel should for small rods be less than the effective depth of the beam, and should bear some relation to the diameter of the bar.

We hope that Sections 56 and 57 will be interpreted to mean, as they should, that splays adjoining columns will be reckoned as increasing the effective depth of beams at this point, so enabling the reverse bending moment to be resisted without providing a very large amount of steel both top and bottom over supports. We therefore suggest that at the end of Section 56 the words be added: "but the depth of the splay may be included in the effective depth of the beam."

In connection with Sections 58 to 63 a requirement should be inserted for distributing bars to be used where the main tensile reinforcement is in one direction only. Such distributing bars are usually a quarter of an inch in diameter and placed about twelve to eighteen inches pitch, but if Clause 60 stands and permits mesh reinforcement to have wires $\frac{1}{10}$ in. diameter, then the distributing bars need not be greater.

In Section 71 (*b*) we would like to see it permitted to take the total distance between the centres of the ribs as the width of the flange in the case of such constructions as hollow-tile floors, but we quite agree with the wisdom of the provision for three-fourths the distance in the case of ordinary secondary or main beam construction.

Section 71 (*c*) should not be permitted to stand, because the width of a tee beam should be solely limited by the pure shearing stress on the concrete along the plane of connection between the rib and the flange, which, as we have previously stated, can be taken at about 50 per cent. of the direct compressive stress.

As regards Section 87, which applies to pillars, we think it unwise to use the term "fixed" for the condition of the ends, because it is seldom that this condition is strictly true. It would be better to use the term "with flat ends," as in American practice, or else "partially fixed," the latter perhaps being more proper to conditions of modern monolithic reinforced concrete construction. The rules for calculating the resistance of pillars seem to us quite serviceable, with the exception of the limiting value of the working stress in Section 102, which would be altogether excessive if the hoops were of large section spaced comparatively wide apart. The spacing of the hoopings has much to do with the limits of the working stress, and we would set these limits as follows, pending a more adequate theoretical analysis than has so far been put forward :—

For rectilinear laterals, $c(1+\cdot 03s)$. For circular hoops, $c(1+\cdot 04s)$.
For cylindrical helices, $c(1+\cdot 05s)$.

Wherein s = spacing factor. The formula for determining the spacing for intermediate values not given in the table should be inserted also.

In Section 105 provision is made for concrete block walls, and we think attention should be directed to the fact that in the near future powers should be obtained to make regulations for reinforced brick walls.

With regard to Section 108, we do not see any reason for limiting the area of openings in walls; it has not necessarily anything to do with the stability of the construction, and if fire risk is the reason for the provision, then a rule should be given which bases the size of the opening upon proximity to adjoining structures. If there is an idea as to the aesthetic value of large wall surfaces, this is perfectly illogical from the point of view of the laws of aesthetics, because there are no canons of proportion other than constructional fitness, and in the case of frame structure the walls may be dispensed with, as in the case of King's College Chapel at Cambridge and many other of the finest specimens of Gothic architecture.

As regards the protection of the steel in reinforced concrete construction, the limits set in Sections 114 to 117 are all very reasonable, except that in the case of pillars we would like to see the alternative given of either increasing the concrete cover or adding a cover of other materials, so that the protection to the steel shall be 4 in. of solid material. Fire tests and experience in actual fires suggest that nothing less is really adequate in buildings and that a good margin of safety is preferable.

In Section 124 it should be noted that "broken hard brick" need not always be satisfactory; some kinds of brick disrupt the concrete, and old bricks should not have any dust or mortar with them, while the greatest care should be paid to seeing that no plaster of Paris, Keene's cement and other sulphates of lime are present.

In Section 127 we do not see why change is made from the ordinary practice of both England and America of regarding sand as all material that will pass a quarter-inch square mesh. The coarser the sand the better, and if there is no idea of reducing the size of what shall be considered sand, with a view to what will be suitable for mortar, we do not see that such a requirement should prevail, for the sort of sand which would be permitted and is usual and quite satisfactory for mortar, is not generally suitable for reinforced concrete work, containing too much loam. Until there is adequate proof that the usual method of taking the limit of a $\frac{1}{4}$ in. for sand is undesirable, it should be left to stand.

In Section 130 there is no need for the 15 per cent. provision as the excess volume of mortar, which should only be slightly in excess of the voids of the coarse material.

Sections 131 and 132 regulate the strength of concrete, and require that the volume of coarse materials in the sand shall bear a definite relation, but we would like to see this altered to a more scientific ruling—namely, to permit scientific proportioning where the voids in the sand and in the aggregate are properly determined, as we fear that such clauses as this might be copied in localities where the materials are different from those likely to be used in the London district. It would be well if the London County Council could give a lead by putting this rule in form adequate to all districts. If the sand is graded in such a manner that the voids are less than 40 per cent., the sand could be more than twice the volume of cement, and in the same way if the voids in the coarse material are less than 40 per cent., the volume of coarse materials could be more than twice the volume of the mortar; it is all a question of grading and the proportion of the voids in the materials. For drafting a scientific rule we would direct attention to Taylor and Thompson's well-known book on concrete, in which such rules are formulated.

In regard to Section 138 steel should never be coated; it is therefore best to delete words having reference thereto. In Section 143 our remarks regarding the provision for deflection apply here also.

In Section 145 a longer time should be allowed before loading tests be applied where the concrete has been exposed to cold and rain. The time should be increased by as many days as frost and rain have intervened.

As to Section 153, neat cement grout should never be used for connecting new concrete to old; mortar should alone be used. Section 157 requires that all structures exposed to frost must be tested; this is undesirable.

The other sections of the regulations are to be highly commended. Some of the practical provisions for the execution of work might be extended, such as by requiring folding wedges at top of all uprights, so that centering can be eased and deflection noted without risk of all collapsing on to the workmen engaged in removing the struts, and possibly others, and wrecking surrounding work, because this is the direction in which the greatest care needs to be taken. Indeed we wish that some such rule could be in force as that in the code of by-laws for this class of construction in Jamaica, where it is required that a superintendent shall be appointed on every job by the municipality or government, and his wages paid by the building owner. In our opinion, supervision by the district surveyor can never be close enough over the practical execution of the work unless an assistant of the district surveyor be always upon the job, and the fees of the district surveyor are totally insufficient to allow of this. Failing such a provision, it is to be hoped that powers will be obtained at any rate to license the foremen engaged on such work. This would prove of advantage to the trade in preventing the adverse opinions of reinforced concrete conjured up by disasters, and would also be of advantage to foremen in increasing their status and by preventing the competition of the incompetent.

The regulations are a distinct step forward in scientific building legislation, and deserve to form a model to other municipalities. The London County Council and their Superintending Architect are to be congratulated on the readiness they have shown in keeping up with the times, as is instanced by the General Powers Act, 1908, permitting greater cubical extent of buildings, the General Powers Act, 1909, recognising steel frame construction, and the present reinforced concrete regulations. Perhaps the Council will later take up the question of reinforced brickwork and a scientific amendment of the existing Acts. The technical law should be in the form of "regulations" which can always be kept up-to-date, and the drafting should be entrusted to the technical advisers of the building authority, and not to the "legal mind" which has made the former Acts so obscure.

HENNEBIQUE FERRO-CONCRETE.

We regret that Messrs. Mouchel & Partners should consider the article which appeared in our March issue entitled "The Revised Report of the Royal Institute of British Architects" to be an attack on their method of design and the safety of Hennebique structures, and we certainly disclaim all intention of implying that their plans are prepared otherwise than in accordance with the usual and recognised methods of calculation employed by the most experienced engineers in reinforced concrete. In our editorial note to the article in question we pointed out that we did not associate ourselves in any way with the views put forward, which we would again repeat. Messrs. Mouchel and Partners have sent us the following letter on the subject :—

March, 1912.

SIR,—Our attention has been drawn to the article by Mr. R. W. Vawdrey in your March issue, professing to discuss "The Revised Report of the Royal Institute of British Architects," but which, with the exception of a few sentences, is nothing more than an attack upon the Hennebique method of design and the safety of Hennebique structures.

We notice that in the introduction to the article you "particularly invite the views of Messrs. Hennebique on this important matter." In response to this invitation we make the following remarks as the representatives of the Hennebique system in the United Kingdom.

Mr. R. W. Vawdrey is a director of the Indented Bar and Concrete Engineering Co., Ltd., a fact that should certainly have been stated by you in order that readers might be able to see at a glance the far from unbiassed position of the writer. We do not propose to say a single word in reply to the article in question, being quite content to leave to your readers the duty of deciding for themselves the impartiality, truth and good taste of the criticisms.—Yours faithfully,

The Editor,
"Concrete and Constructional Engineering."

For L. G. MOUCHEL & PARTNERS,
J. S. E. DE VESIAN, Director.

REINFORCED CONCRETE GANTRY AT HARRINGTON COLLIERY.

We have on several occasions pointed out the advisability of using reinforced concrete in colliery work, and we are therefore glad to be able to publish in this issue a description of some very interesting work in that material at Harrington Colliery. The article was prepared for us by Mr. Albert Lakeman, Hon. Medallist Construction.—ED.

This example provides us with another instance of the extensive use of reinforced concrete, which is being employed for all classes of construction, and its adaptability is admirably illustrated in work of this kind.

The scheme consists of a gantry for the conveyance of coal from the pit head to the screening house, and from thence to the railway siding, and the work will be more clearly understood if some idea of the system employed is first gathered.

The general plan is illustrated in *Fig.* 1, and it will be seen that the pit head is situated in the centre of the banking house, and it is at this point that the coal is raised in the trucks, which are lifted from the mine in a two-storied cage. This banking house has a width of 26 ft. and a length of about 80 ft., and an upper and lower runway are constructed to suit the two levels of the cage in which the trucks are raised. These runways are provided with rails, and upon these the trucks run by gravity for a distance of about 80 ft., where the gantry is widened out to allow two sets of rails to run side by side, and creepers are provided which raise the trucks to such a height as will give the necessary fall to the screening house. Owing to the difference in the level of the rails when leaving the banking house, the two creepers are of different lengths, in order to bring the trucks to the same level and allow them to enter the screening house at the same point. Upon reaching the screening house the trucks are spread on separate lines of rails, each of which is provided with a "tippler," consisting of a vertical wheel in which the truck is caught and made to rotate in such a manner as to empty the contents on to the screens situated on the floor below. After the trucks are emptied they assume their normal position, and leave the "tippler" to proceed on the return journey to the banking house. As the trucks leave the screening house on the same level, a creeper is necessary on the return gantry for raising one half of the trucks in order that they may reach the high-level floor of the cage by which they are returned to the mine. All these varying levels naturally complicated the design of the gantries, which also required to be of varying widths to suit one or two lines of rails, as the case might be, 9 ft. being required in the former case, and 13 ft. in the latter.

In addition to the truck gantry, a belt conveyor gantry 6 ft. wide and about

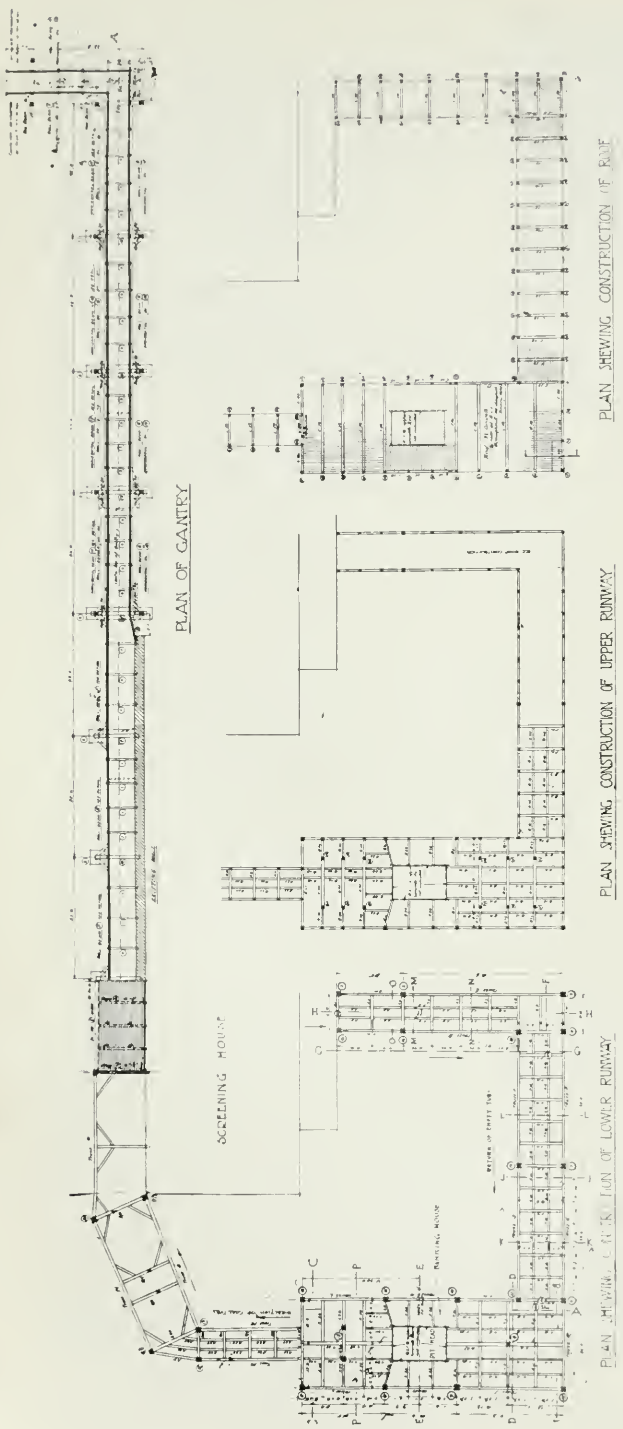

PLAN OF GANTRY
PLAN SHEWING CONSTRUCTION OF ROOF
PLAN SHEWING CONSTRUCTION OF UPPER RUNWAY
PLAN SHEWING CONSTRUCTION OF LOWER RUNWAY
SCREENING HOUSE
BANKING HOUSE

Fig. 1. Plan.
Reinforced Concrete Gantry at Harrington Colliery.

280 ft. long is attached to the screening house for conveying the coal direct to the railway trucks. A small building was formed between the truck gantry and the belt conveyor gantry as a repairing house for the trucks.

The columns to the banking house are spaced at distances varying from 20 ft. 9 in. to 31 ft. 3 in., and these are connected by trusses as shown in *Fig.* 2, which is a detail section of the banking house. The size of these columns at the base varied from 15 in. by 15 in. to 18 in. by 18 in., and the reinforcement, generally speaking, consisted of four $\frac{3}{4}$-in. diameter rods with $\frac{3}{16}$-in. binding at 6-in. centres.

The foundations consisted of a reinforced concrete slab with stiffening beams running out from the columns to the edges of the slab in the form of a cross, the size of the slab varying from 4 ft. 6 in. under the smallest column to 6 ft. 6 in. under the largest, with reinforcement consisting of $\frac{3}{8}$-in. bars at 6-in. centres, with Moss bars in the main ribs.

A typical truss is that illustrated in *Fig.* 2, where it will be

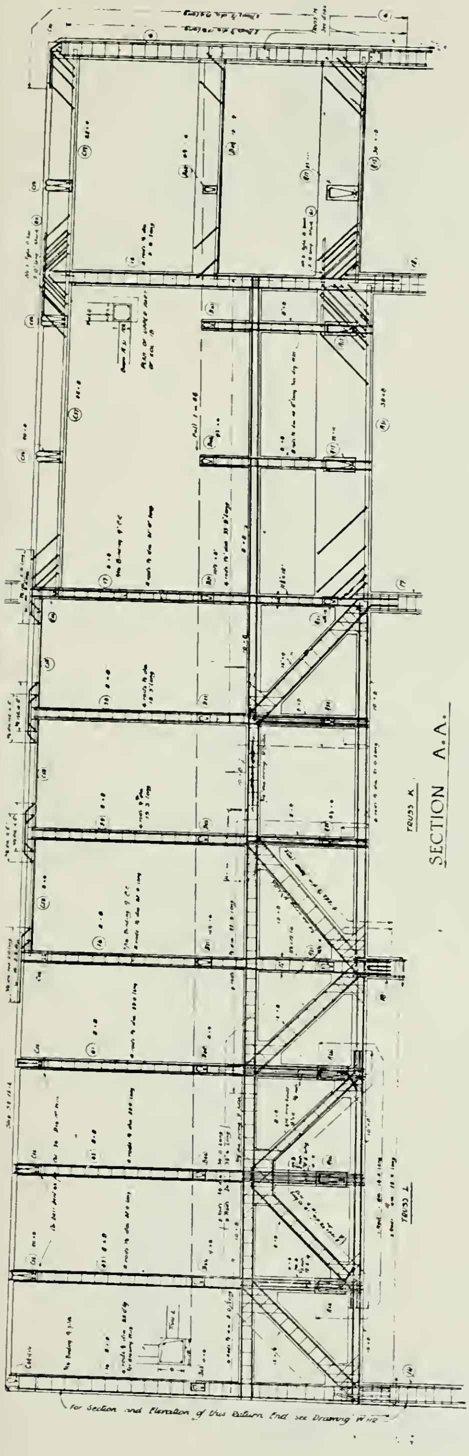


Fig. 2. Section of Banking House.
Reinforced Concrete Gantry at Harrington Colliery.

seen that the beams which carry the rails are framed into vertical members spaced at about 7-ft. intervals, these members varying from 8 in. square to 10 in. by 8 in., with similar bracing members as shown in the drawing. The reinforcement in most cases consists of four $\frac{1}{2}$-in. bars with $\frac{3}{16}$-in. binding at 9-in. centres, while the cross-beams carrying the rails varied considerably in size, according to the span, and were reinforced with Moss bars and stirrups. The lower set of cross-beams had an average height from the ground of 21 ft., while the total height of the banking house is about 38 ft., and as the space between the constructional members was filled in with expanded metal and cement plaster, a great deal of wind pressure had to be resisted.

The drawing illustrated in *Fig.* 3 shows an instance where wind bracing was provided, this being a section through one point in the return gantry giving only the lower part below the level of the rails. The size of the cage is shown in *Fig.* 4, which is a cross section through the lower part of the banking house, and it will be seen how the columns are inclined inwards and braced with an intermediate strut 12 in. by 8 in., reinforced with four $\frac{5}{8}$-in. diameter rods and $\frac{3}{16}$-in. wire ties at 9-in. centres. The truss L which is indicated in this illustration is shown in elevation in *Fig.* 2.

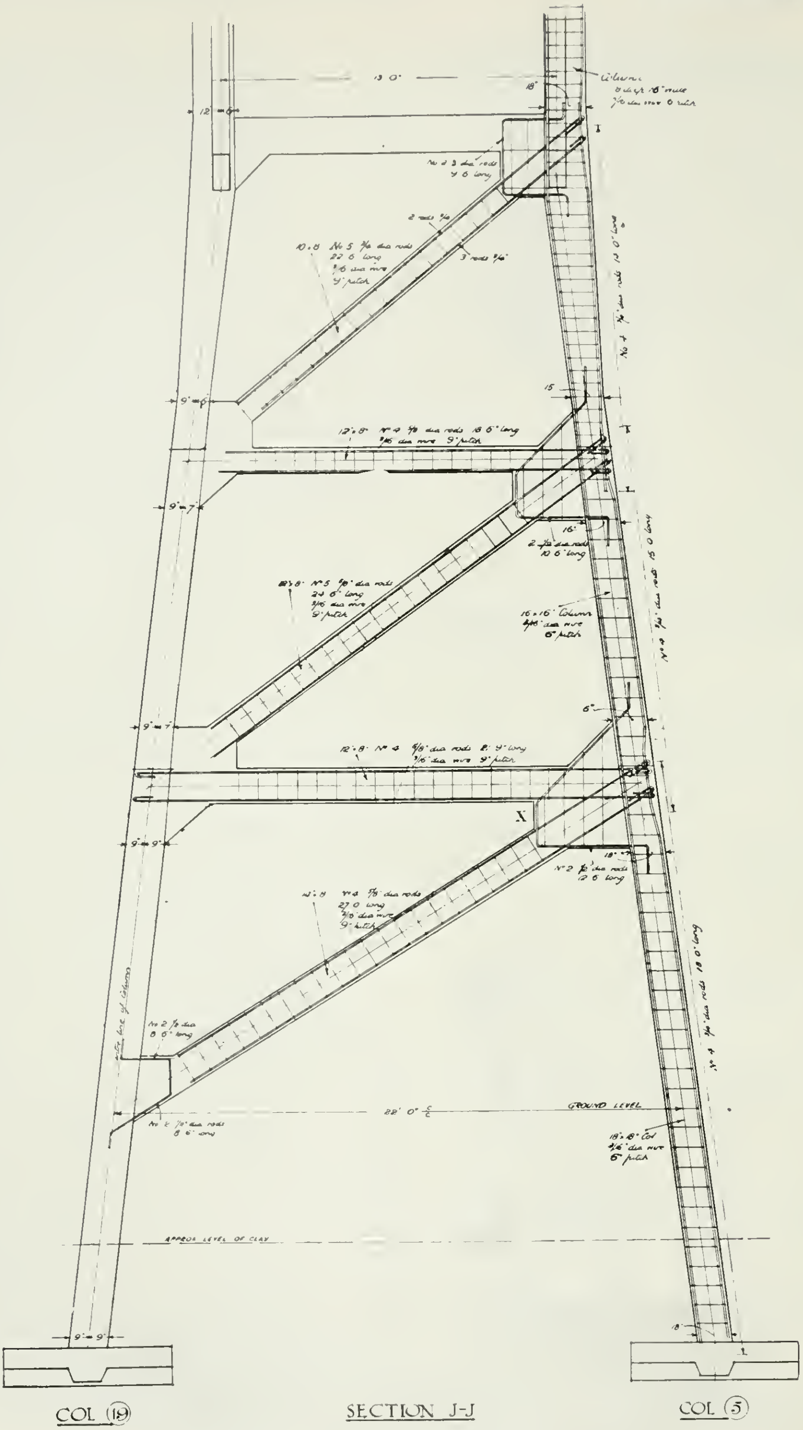

13' 0"
12" 6"
18"
Column 18" sq. 18" rods 7/16 dia. wire 6" pitch
No. 2 3/4 dia. rods 4' 6" long
2 rods 7/8
3 rods 7/8"
10 × 8 No. 5 7/8 dia. rods 22' 6" long 3/16 dia. wire 9" pitch
No. 4 7/8 dia. rods 13' 0" long
9" 6"
15
12" × 8" No. 4 7/8 dia. rods 18' 6" long 3/16 dia. wire 9" pitch
9" 7"
16"
2 7/8 dia. rods 10' 6" long
12 × 8" No. 5 7/8" dia. rods 24' 6" long 3/16 dia. wire 9" pitch
16 × 16" Column 3/16 dia. wire 6" pitch
No. 4 7/8" dia. rods 15' 0" long
6"
9" 7"
12" × 8" No. 4 5/8" dia. rods 21' 9" long 3/16 dia. wire 9" pitch
X
9" 9"
18"
No. 2 7/8 dia. rods 12' 6" long
12 × 8 No. 4 7/8" dia. rods 27' 0" long 3/16" dia. wire 9" pitch
No. 4 7/8 dia. rods 18' 0" long
Centre line of Columns
No. 2 7/8 dia. 8' 6" long
88' 0" C/C
GROUND LEVEL
No. 2 7/8" dia. rods 8' 6" long
18" × 18" Col 3/16" dia. wire 6" pitch
APPROX. LEVEL OF CLAY
9" 9"
18"
COL ⑲
SECTION J-J
COL ⑤

Fig. 3. Section showing Wind Bracing.
Reinforced Concrete Gantry at Harrington Colliery.

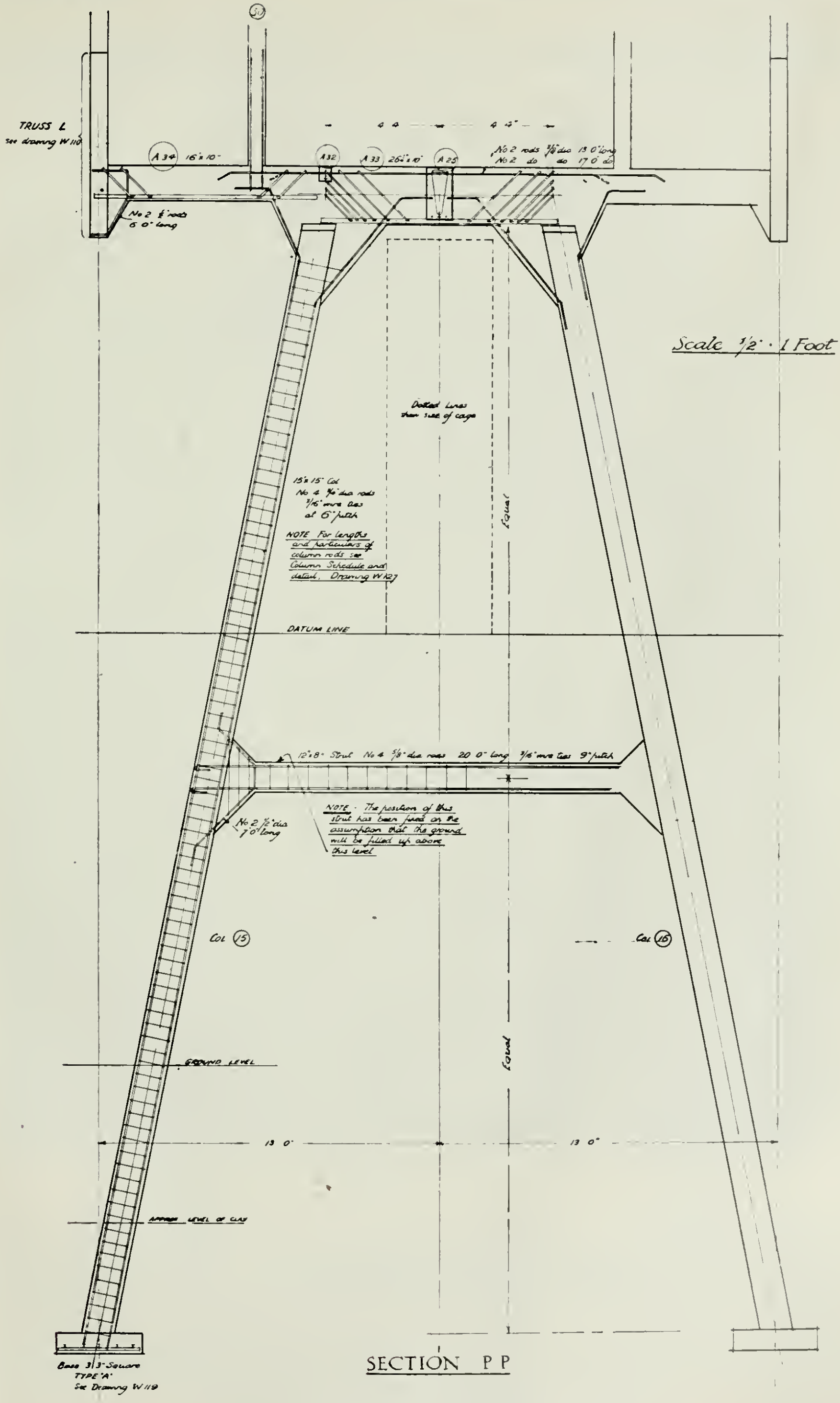

50
TRUSS L
see drawing W 110
A 34 16" x 10"
A 32
A 33 26½" x 10"
A 25
4 4" 4 4"
No 2 rods ⅞" dia 13 0" long
No 2 do do 17 0" do
No 2 ⅞" rods 6 0" long
Scale ½" · 1 Foot
Dotted Lines show size of cage
15" x 15" Col
No 4 ⅞" dia rods
3/16" wire ties
at 6" pitch
NOTE For lengths and particulars of column rods see Column Schedule and detail, Drawing W 127
Equal
DATUM LINE
12" x 8" Strut No 4 ⅝" dia rods 20 0" long 3/16" wire ties 9" pitch
No 2 ½" dia 7 0" long
NOTE · The position of this strut has been fixed on the assumption that the ground will be filled up above this level
Col 15
Col 16
Equal
GROUND LEVEL
13 0"
13 0"
APPROX LEVEL OF CLAY
Base 3' 3" Square
TYPE 'A'
See Drawing W 119
SECTION P P

Fig. 4. Cross Section through Banking House.
Reinforced Concrete Gantry at Harrington Colliery.

It will be seen that these two columns are situated inside the banking house as shown in the plan illustrated in *Fig.* 1, and are distinct from those main

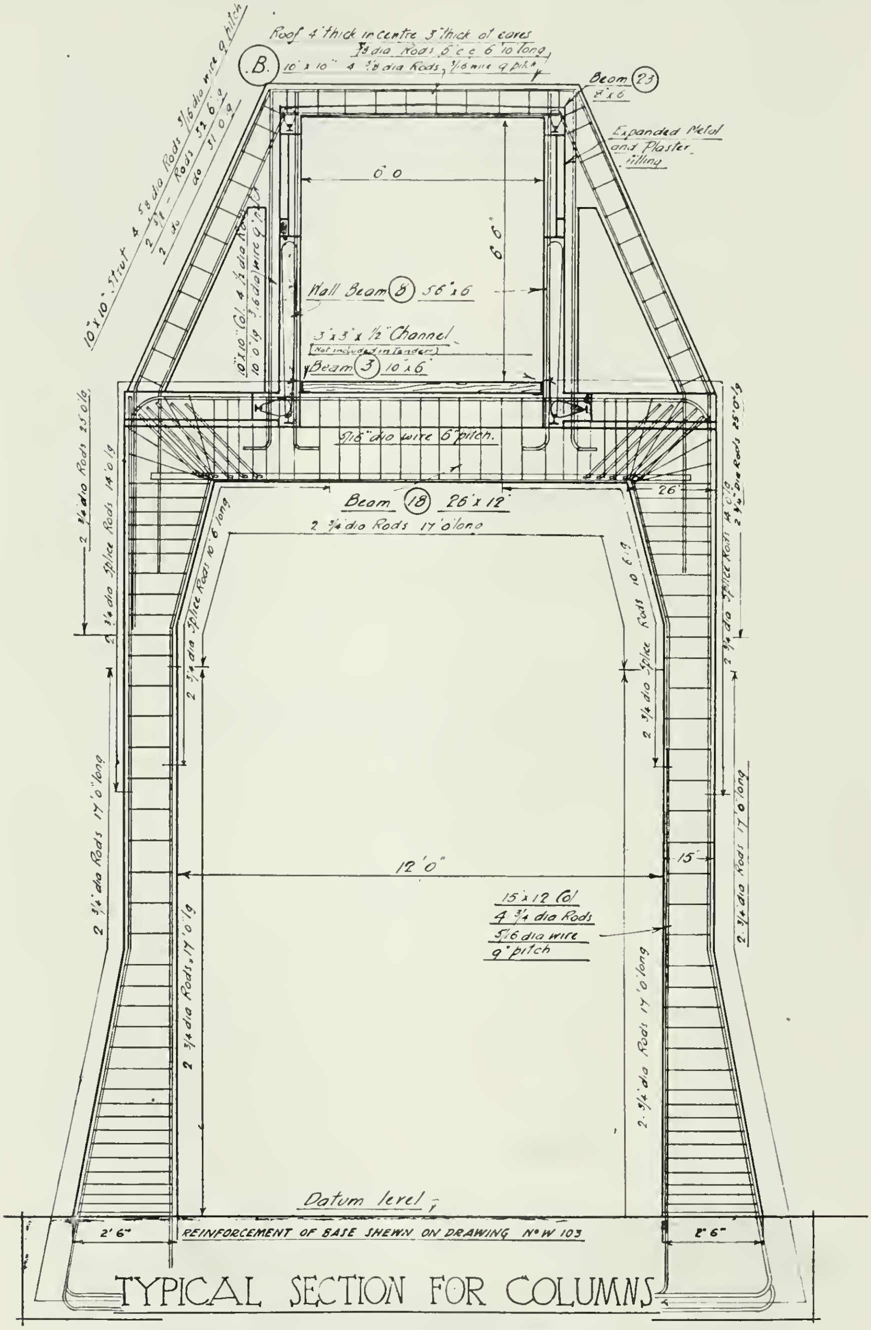
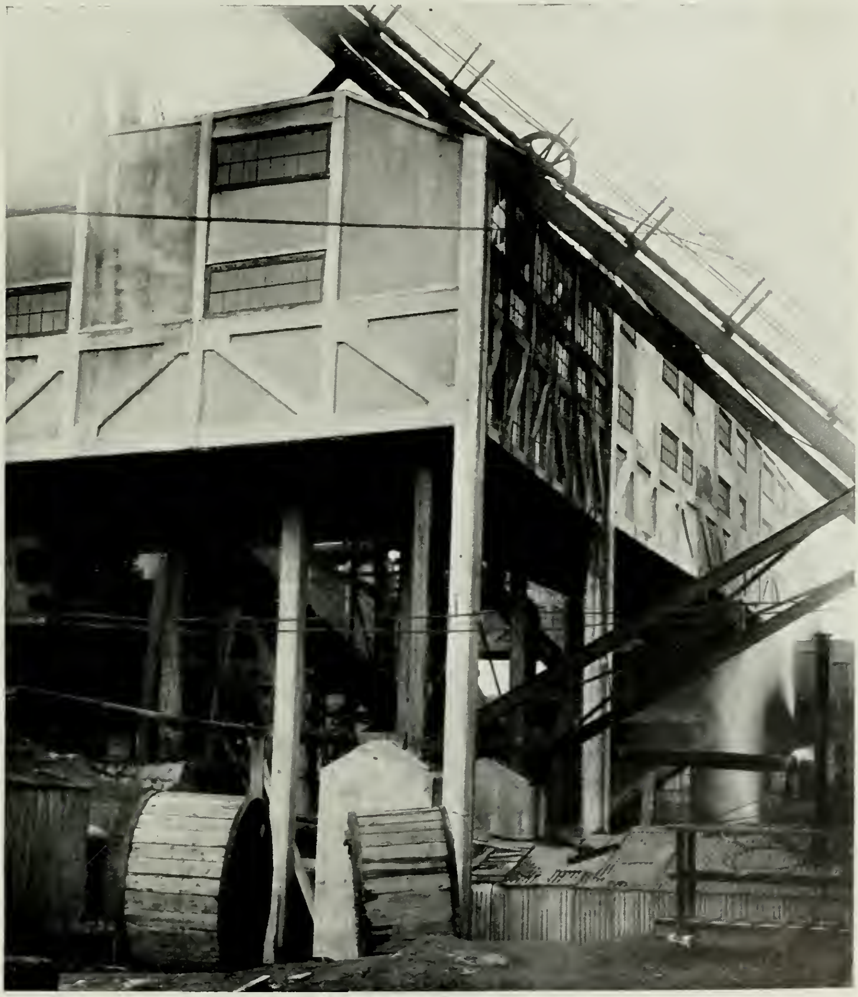

Roof 4" thick in centre 3" thick at eaves
5/8 dia Rods 5" c c 6' 10" long
B.
10" x 10" 4 3/8 dia Rods, 5/16 wire 9" pitch
Beam 23
8" x 6
Expanded Metal and Plaster filling
10" x 10" Strut 4 5/8 dia Rods 5/16 dia wire 9" pitch
2 5/8 - Rods 5' 6" o/g
2 do do 5' 0" o/g
6' 0
6' 6"
10" x 10" Col. 4 1/2 dia Rods 5/16 dia wire 9" pitch
10' 0" o/g
Wall Beam 8 56" x 6
3" x 3" x 1/2" Channel
(Not included in Tender)
Beam 3 10" x 6"
5/16" dia wire 6" pitch.
2 3/4 dia Rods 25' 0" o/g
2 3/4 dia Splice Rods 14' 0" o/g
Beam 18 26" x 12"
26"
2 3/4 dia Rods 17' 0" long
2 3/4 dia Splice Rods 10' 6" long
2 3/4 dia Splice Rods 10' 6" o/g
2 3/4 dia Splice Rods 14' 0" o/g
2 3/4" dia Rods 25' 0" o/g
12' 0"
15"
15" x 12" Col
4 3/4 dia Rods
5/16 dia wire
9" pitch
2 3/4 dia Rods 17' 0" long
2 3/4 dia Rods, 17' 0" o/g
2 3/4 dia Rods 17' 0" long
2 3/4 dia Rods 17' 0" long
Datum level
2' 6"
REINFORCEMENT OF BASE SHEWN ON DRAWING No W 103
2' 6"
TYPICAL SECTION FOR COLUMNS

Fig. 5. Cross Section of Belt Conveyor Gantry
REINFORCED CONCRETE GANTRY AT HARRINGTON COLLIERY.

columns previously described which carry the ends of the trusses. The roof of the banking house was formed with a reinforced slab 3½ in. thick between the cross-beams which connect the upper columns at the head, these cross-beams being reinforced with Moss bars and stirrups.

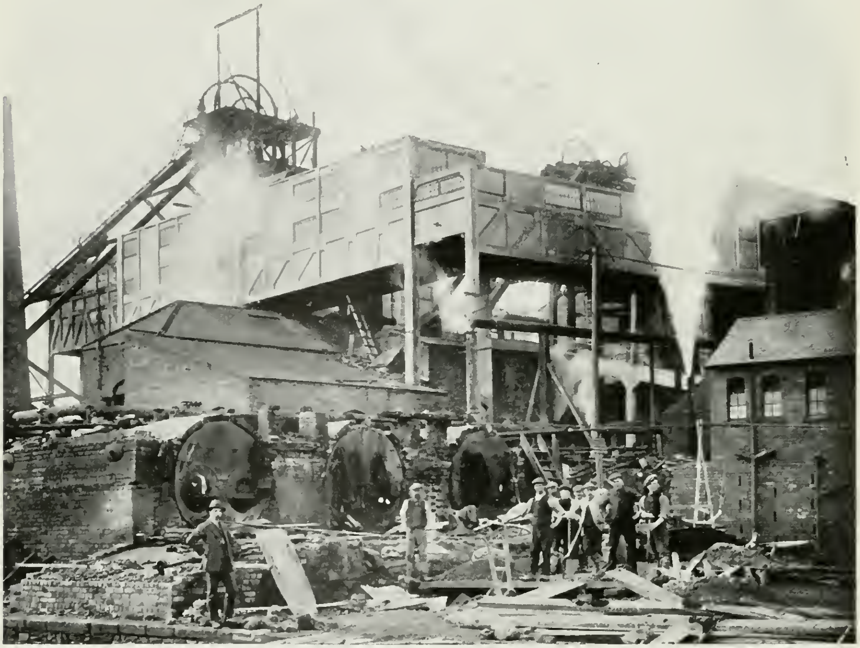
The repairing shop was provided with a sloping roof formed of a 3½-in. slab reinforced with ½-in. diameter rods at 10-in. centres, with skylight openings between the purlin beams, these latter having an average size of 13 in. by 8 in.



View of Tub Gantry.
Reinforced Concrete Gantry at Harrington Colliery.

The belt conveyor gantry previously mentioned affords some interesting construction, owing to its great length. The general spacing of the columns was 40 ft., and the diagram in *Fig.* 5 shows a typical cross section of the gantry, where it will be observed that no bracing was provided in this case between the columns, as the clear space was necessary owing to circumstances. The general system and size of the reinforcement are sufficiently indicated to need no explanation.

Some interesting construction, which is not fully illustrated in the diagrams reproduced, was provided by the wall beams on this gantry, which had a span of 40 ft. These were 56 in. deep and 6 in. thick, formed with a 14-in. by 8-in. horizontal stiffener beam at the bottom, and with vertical struts at 8-ft. centres 8 in. by 6 in., reinforced with four $\frac{1}{2}$-in. diameter rods and $\frac{3}{16}$-in. wiring at 9-in. pitch. Continuity bars 14 ft. long were provided over the main columns, and Moss bars and stirrups were provided as the main reinforcement.

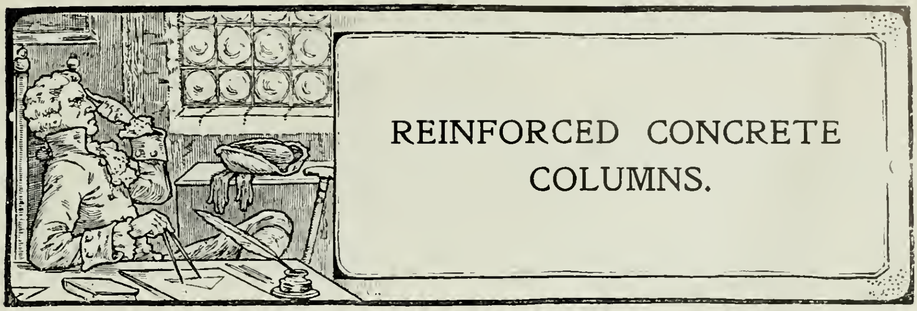


Another View of Tub Gantry.
Reinforced Concrete Gantry at Harrington Colliery.

Small cross-beams were formed at intervals of about 8 ft. between the main beams connecting the columns to carry the floor, and the roof was constructed of a reinforced slab varying from 4 in. thick in the centre to 3 in. thick at the eaves, and $\frac{3}{8}$-in. diameter rods at 6-in. centres.

A great deal of very interesting work can be gathered from the drawings of this example of reinforced concrete work, which afforded certain complications to the designer which are not met with in ordinary building work; and no other material would have lent itself so readily to the different conditions.

The whole of the reinforced concrete work was executed by Messrs. William Moss and Son, Ltd., Queen Anne's Chambers, Victoria Street, from designs prepared under the supervision of their chief engineer, Mr. H. de Colleville, A.R.I.B.A. The work was carried out for Mr. Cecil Walton, of Moresby, Westmorland.



REINFORCED CONCRETE COLUMNS.

We publish below an article on reinforced concrete columns by W. E. Lilly Professor of Engineering at Trinity College, Dublin, which merits attention.—ED.

By W. E. LILLY, D.Sc.

In this article it is proposed to discuss the derivation of a formula for determining the strength of reinforced concrete columns which is generally applicable to the majority of cases that arise in practice. The formula referred to is a modified form of the Rankine-Gordon formula for columns, the derivation of which will first be given.

Consider the column shown on *Fig.* 1 to be of uniform cross-section and assume that the column is slightly bent into a parabolic curve by the applied load P, then the bending moment at the middle is

$$M = Pd = pAd \qquad (1)$$

where P denotes the load on the column,
d ,, the central deflection,
p ,, the load per unit area,
A ,, the area of the cross-section.

From the theory of bending the following relation obtains for the equilibrium of the central cross-section:

$$\frac{M}{I} = \frac{p_1}{y} \qquad (2)$$

where I denotes the moment of inertia of the cross-section about a diameter,
y ,, the distance of the outer fibre from the neutral axis,
P_1 ,, the intensity of the outer fibre stress due to bending.

Also $I = A\rho^2$ where ρ is the radius of gyration of the cross-section. From equations (1) and (2) the following relation obtains:

$$pAd = \frac{p_1 I}{y} \text{ or } \frac{p_1}{p} = \frac{dy}{\rho^2} \qquad (3)$$

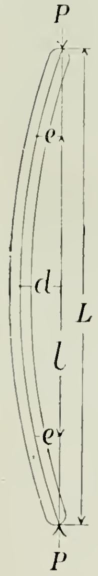


FIG. 1.

If the central cross-section is considered as being made up of two areas of which A_1 transmits the direct stress and the other A_2 the bending stress as shown on *Fig.* 2, and the direct stress p on A_1 is put equal to the bending stress p_1 on A_2 then equation 3 becomes

$$p_1 A_1 d = \frac{p_1 A_2 \rho_2}{y} \quad \text{or} \quad \frac{A_2}{A_1} = \frac{dy}{\rho_2} = \frac{p_1}{p} \qquad (4)$$

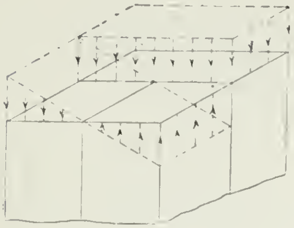


FIG. 2.

Adding unity to both sides of this equation and putting $p + p_1$ equal to f the strength to compression of the material, then

$$P = pA = \frac{fA}{1 + \frac{dy}{\rho^2}}. \qquad (5)$$

In deriving the above result it is tacitly assumed that the strength to compression of the material is equal to its strength to tension. In order to modify it to include materials in which the strength to compression f_k is different from the strength to tension f_t, equation 4 becomes

$$f_k A_1 d = \frac{f_t A_2 \rho^2}{y} \quad \text{or} \quad \frac{A_2}{A_1} = \frac{f_k dy}{f_t \rho^2}$$

and (6)

$$P = f_k A_1 = \frac{f_k A}{1 + \frac{f_k dy}{f_t \rho^2}}$$

The result obtained in equation 6 is only a close approximation and differs in this respect from the result of equation 5, which is rigorously correct on the usually accepted theory of bending.

In deriving the Rankine-Gordon formula the deflection of the column is given by the expression

$$d = \frac{fL^2}{\pi^2 E y} \qquad (7)$$

where L is the length of the column, E Young's modulus of elasticity, and f the strength to tension or compression for the material when these strengths are equal, for materials in which the strength to compression is greater than the strength to tension the value of f is put equal to f_t, the strength to tension.

The relation given in equation 7 is the same as that derived from the theory of bending for a beam resting on two supports and loaded proportionally to the ordinates of a sine curve. For small deflections the deflection curve is sinusoidal and closely approximates to the parabola.

Now assume the deflection curve of the bent column shown on *Fig.* 1 to be a parabola, then

$$d = \frac{fL^2}{\pi^2 E y} = e + \frac{fl^2}{\pi^2 E y}$$

and (8)

$$\frac{dy}{\rho^2} = \frac{ey}{\rho^2} + \frac{fl^2}{\pi^2 E \rho^2}$$

where e is the eccentricity of loading of the column and equal to the distance of the load from the centre of the ends of the column. Substituting from equation 8 in equation 5 gives

$$P = \frac{fA}{1 + \frac{ey}{\rho^2} + \frac{fl^2}{\pi^2 E \rho^2}} \qquad (9)$$

which is the form of the Rankine-Gordon formula, modified to include eccentric loading, and applies to those materials in which the strengths to compression and tension are equal.

To further modify the above formula to include those materials in which the strengths to compression and tension f_k and f_t are unequal, multiply equation 8 by f_k divided by f_t then

$$\frac{f_k dy}{f_t \rho^2} = \frac{f_k ey}{f_t \rho^2} + \frac{f_k l^2}{\pi^2 E \rho^2} \qquad (10)$$

Substituting from equation 10 in equation 6 gives

$$P = \frac{f_k A}{I + \frac{f_k ey}{f_t \rho^2} + \frac{f_k l^2}{\pi^2 E \rho^2}} \qquad (11)$$

From the author's experiments* on columns, the value of the ratio f_k divided by f_t is approximately 3 for cast iron. The results of a large number of experiments on columns of this material showed that the formula could be relied upon to determine with a fair degree of accuracy the strength of these columns and in all cases erred on the side of safety.

The application of the formula to reinforced concrete will now be considered. On *Fig.* 3 is shown a short reinforced concrete prism under a direct compressive stress. Let E_c and E_s be respectively Young's modulus of elasticity for concrete and mild steel. Assume when the load P is applied to the prism of length H that the decrease in the length h is the same for both concrete and steel, then by Hooke's law

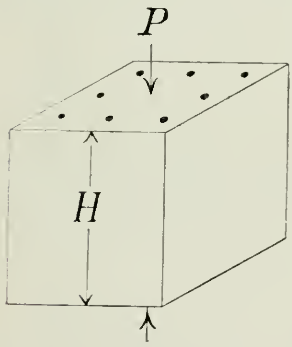


Fig. 3.

$$\frac{P_c}{E_c A_c} = \frac{P_s}{E_s A_s} = \frac{h}{H} \qquad (12)$$

where P_c and P_s are respectively the loads on the concrete and steel and A_c and A_s the areas of the cross-sections. Consider now the reinforced concrete prism to be made up of a substance in which the longitudinal reinforcement is uniformly distributed throughout its cross-section, then Young's modulus of elasticity E and the load P for this substance would be given by the relation

$$\frac{P}{EA} = \frac{P_c}{E_c A_c} = \frac{P_s}{E_s A_s}$$

Also $P = P_c + P_s$ and $A = A_c + A_s$, giving (13)

$$E = \frac{E_c A_c + E_s A_s}{A}$$

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* Proc. Inst.C.E., Vol. clxxxi.

Recent experiments* on reinforced concrete columns show that the equal shortening of the concrete and steel actually takes place and therefore the value obtained for E is closely correct.

Consider now the strength to compression of the reinforced concrete prism shown on *Fig.* 3. Let f_c and f_t be respectively the strengths to compression of the concrete and steel, then

$$P_c = f_c A_c \text{ and } P_s = f_s A_s$$

Let $P = P_c + P_s = f_k A$, where f_k represents the strength to compression of the substance made up of the concrete and steel, and A the area of its cross-section, then $P = P_c + P_s = f_k A = f_c A_c + f_s A_s$ and $f_k = \frac{f_c A_c + f_s A_s}{A}$ (14)

A similar expression to that already obtained for E. Experiments also show that approximately the ratio

$$\frac{f_c}{E_c} = \frac{f_s}{E_s} = \text{a constant.}$$

For instance, for mild steel $E_s = 30000000$ and f_s about 60000, and for concrete $E_c = 1500000$ and f_c about 3000, giving a value of the constant about 1/500. Assuming the above relation and substituting in equations 13 and 14 gives

$$\frac{f_k}{E} = \frac{f_c A_c + f_s A_s}{E_c A_c + E_s A_s} = \frac{1}{500} \qquad (15)$$

The value thus obtained for the constant is an average approximation; for any particular example when the terms in equation 15 are known the constant can easily be determined.

It remains now to evaluate the ratio f_k divided by f_t; for reinforced concrete beams it is not unusual to assume that the concrete does not transmit any tensile stress: this gives from equation 14

$$f_t = \frac{f_s A_s}{A} \text{ and } \frac{f_k}{f_t} = \frac{f_c A_c + f_s A_s}{f_s A_s} \qquad (16)$$

In determining the ratio between the areas of the concrete and steel for columns $f_c A_c$ is made approximately equal to $f_s A_s$, giving a value of f_k, divided by f_t of about 2. Reverting now to the formula given in equation 11 with the constants as determined it may be written

$$P = \frac{f_c A_c + f_s A_s}{1 + 2\frac{ey}{\rho^2} + \frac{1}{5000}\frac{l^2}{\rho^2}} \qquad (17)$$

The formula so obtained shows that the strength of the reinforced concrete column is proportional to the sum of the strengths of the longitudinal steel reinforcement and concrete. For any particular case the constants can be determined as already shown; a considerable variation in the proportions of the materials makes but little difference in the values of these constants. In deriving the formula it is assumed that the radius of gyration of the longitudinal steel reinforcement is the same as that of the cross-section of the concrete about the same neutral axis; the design of the column always involves the placing of the steel as far from the centre of the column as practicable, and therefore this condition is closely fulfilled. For columns in which spiral steel reinforcement is used, the effect of the reinforcement is only to be considered as strengthening the concrete to resist compression, and therefore is to be taken into consideration when the value of f_c is being evaluated; the spiral reinforcement has but little or no effect in strengthening the column against bending.

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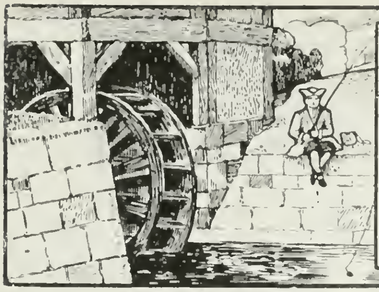
* Proc. Inst. C.E., 1912.

In the majority of reinforced columns the length of the column divided by the side is about 10 to 16 and the value of the length divided by the radius of gyration for these columns is about 30 to 50, and therefore the value of the third term in the denominator of the formula for these columns is about $\frac{1}{5}$ to $\frac{1}{2}$.

In using the formula the value of f_s the strength to compression of mild steel should be taken about 30,000 pounds per square inch, that is the strength near the yield point of the material, as the bond between the concrete and steel cannot be assumed to hold beyond this value. If the longitudinal reinforcement is not parallel to the axis of the column—that is, if it is at all bent or buckled—a still lower value must be adopted.

The longitudinal reinforcement has been assumed to be connected to the transverse reinforcement; the effect of this transverse reinforcement and the allowance to be made for it on the strength of the column has been, except in the case of the spirally wound reinforcement, neglected.



REINFORCED CONCRETE RESERVOIRS AT MT. TABOR, PORTLAND, OREGON.

The description given below of two large reinforced concrete reservoirs in Portland, Oregon, is a further proof of the suitability of this method of construction for works of this nature—such as reservoirs, storage tanks, water towers, etc.—ED.

Two new reinforced concrete reservoirs and a new steel conduit are being built at Portland, Oregon, which, when completed, will triple the present supply and storage. The reservoirs are on Mt. Tabor, near the eastern limits of the city.

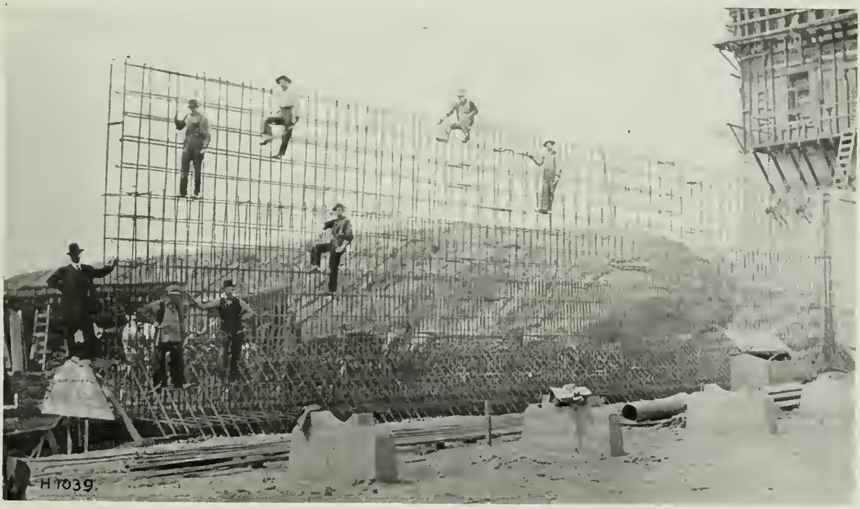
The upper reservoir has an oddly shaped curved rim, 2,100 ft. long, a water surface of 6 acres, a maximum depth of water of 32 ft., and a capacity of 50 million gallons. The lower reservoir, 107 ft. below the upper one, is rectangular in shape, has a water surface of 12 acres, a maximum depth of water of 22 ft., and a capacity of 75 million gallons.

Construction joint in aqueduct.
REINFORCED CONCRETE RESERVOIRS AT MT. TABOR, PORTLAND, OREGON.

The general character of the work is the same on both reservoirs, varying in details. Each reservoir is on sloping ground, a part of the excavation from the upper side forming a bank at the lower side, and the basin being lined with concrete.

Upper Reservoir.—This reservoir is in a natural draw, and a dam, consisting of a vertical reinforced concrete wall backed by a rolled earth embankment, was built across the mouth of this draw. This wall is of counterfort type, 360 ft. long, with a maximum height of 69 ft., of which 32 ft. is below the level of the reservoir floor. The gate chamber which projects into the basin from the face of the dam, the pipe conduit under the rolled embankment, and the aqueduct supported on the counterforts of the wall are all of reinforced concrete and were built up with and bonded into the wall.

In the design of these structures 500 lb. per sq. in. was allowed for compression in concrete and the tensile strength of same was ignored. 16,000 lb. per sq. in. was allowed as the tensile strength of the reinforcing steel. Except in the aqueduct the area of reinforcement was ·65 of 1 per cent. In the aqueduct, which was reinforced for both external and internal pressures, the area of steel was about 1 per cent. Plain round bars were used and the ends were hooked and lapped thirty diameters.



Steel in place for one section of division wall.
Reinforced Concrete Reservoirs at Mt. Tabor, Portland, Oregon.

For economy in construction one vertical joint was made in the wall and aqueduct. In the wall joint extra heavy reinforcement was carried through, and half of a 12 in. L beam embedded in each side of the joint. In the aqueduct joint extra steel was carried through and a strip of sheet lead bent in the form of a channel beam with a wrinkle in the flange was built in to insure the joint against leaks. No attempt was made to pour continuously, the usual dovetailing and grouting being done at each horizontal joint.

The lining is plain concrete 9 in. thick on the bottom and tapered to ·5 in. at the top of the slope, which is 1 on 2. It is built in slabs 15 ft. by 20 ft. with tarred felt and asphalt joints. Each joint is supported by a concrete beam 8 in. by 6 in.

The top of the slab is covered with one-half inch of plaster, applied before concrete has its initial set. The plaster is floated and trowelled smooth. Around the rim is a concrete parapet which will support an ornamental iron fence. This parapet has longitudinal reinforcing steel and was built in sections about 30 ft. long.

Outside the parapet is a concrete walk and on the top of the dam a driveway will form a part of the park system.

The sand and gravel were dredged from the rivers near the city, and, together with all other supplies and machinery were delivered at the works by the local electric railway company. All gravel would pass through a 2-in. screen and averaged 37 per cent. voids. The voids in sand averaged 34 per cent.

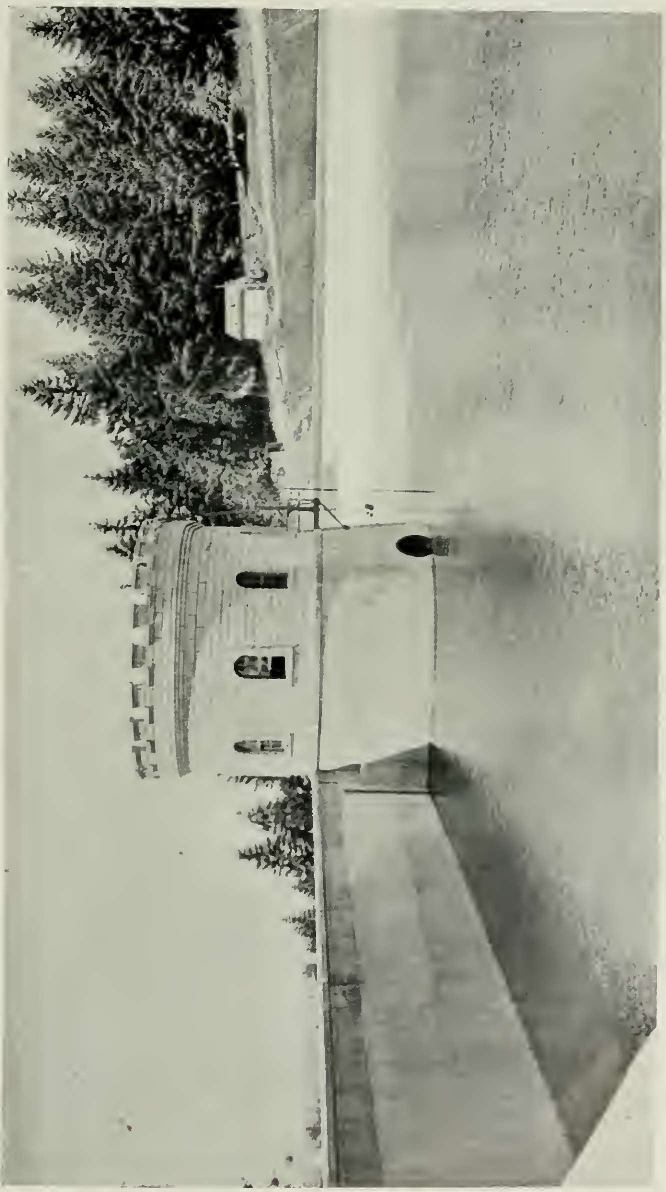
Work in course of construct.on.
Reinforced Concrete Reservoirs at Mt. Tabor, Portland, Oregon.

The proportions of mix were 1 : 3 : 6 for foundations and 1 : 2 : 4 for all other concrete with 1 : 2 for plaster. All concrete was mixed in machine mixers driven by electric motors.

During 1910 tests of various waterproofing compounds were made. Samples containing some of the patent compounds were found to 'be waterproof, but some samples of plain concrete and plaster were also waterproof. The result of these tests was the abandonment of any waterproofing, reliance being placed on the density of the concrete and plaster to prevent seepage. And at the time of writing, with 10 ft. of water in the basin it is absolutely tight. Underdrains discharging into a sump in the gate chamber allow inspection and measurement of ground water and seepage.

After the forms were removed all water surfaces were plastered. The parapet was not plastered, but extra care was taken with the forms, which were removed as soon as the concrete would stand, usually 12 hours after pouring. After forms were removed the concrete surface was smoothed with a steel trowel. The parapet, as well as all other exposed concrete, was kept wet for a week after pouring.

On the outside of the gate-house superstructure joint strips in the forms marked the surface into rectangles, and in pouring a thin coat of mortar was plastered against the forms ahead of the concrete. These forms were removed one month after pouring and the surface was then tooled with a pneumatic bush hammer.



View of gate-house and a portion of completed reservoir.

Reinforced Concrete Reservoirs at Mt. Tabor, Portland, Oregon.

Lower Reservoir. —This reservoir is 800 ft. west of and 107 ft. below the upper reservoir. Here the basin was completed by building a rolled earth embankment on one side and one end and placing the concrete lining on the slope of this embankment.

The reservoir is divided into two basins by a reinforced concrete division wall and has an inlet and an outlet gate chamber with pipes in duplicate.

The division wall is of cantilever type, with expansion joints 85 ft. apart. These joints are tongued and grooved and made watertight with sheet lead and asphalt. The exterior finish of the concrete is similar to that at the upper reservoir.

In both reservoirs together there will be 35,000 cubic yards concrete and 850,000 lbs. of reinforcing steel.

The whole of this work was designed and superintended by Mr. Edwin A. Taylor, of Portland, Oregon, U.S.A.

RECENT BRITISH PATENTS RELATING TO CONCRETE.

We propose to present at intervals particulars of British Patents issued in connection with concrete and reinforced concrete. These particulars have been prepared for this Journal by Mr. A. W. Farnsworth, of Strand Chambers, Derby. ED.

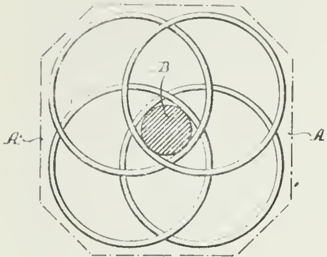


FIG. 1.

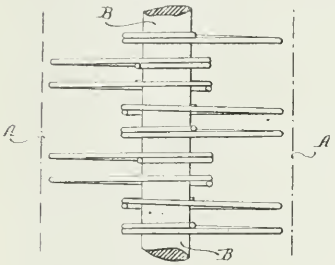
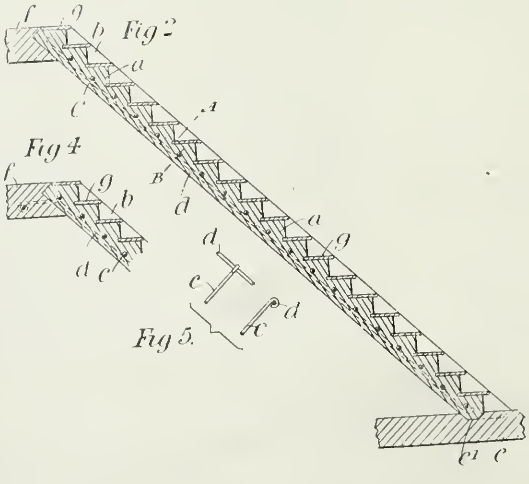
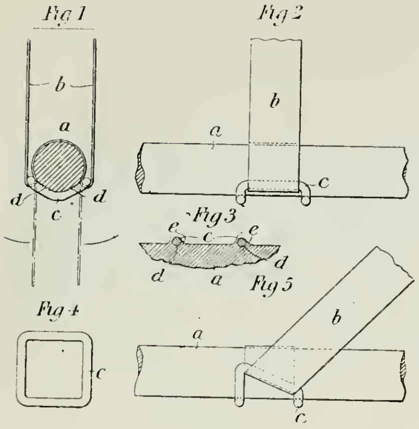


FIG. 2.

Column Reinforcement.—No. 27,528/10. *Thomson and Thomson. Accepted November 9/11.*—This is a modification of the spiral reinforcement which is intended to avoid the use of several vertical bars. For the familiar spiral are substituted a series of steel rings which are embedded in the concrete as the work proceeds; they are laid horizontally and eccentrically about a single central steel rod. It is explained by the inventors that this disposition of the reinforcement enables the rings to exert a mutually balancing pull to resist the tendency of the concrete to bulge laterally under an excessive load. It is claimed that the stresses produced in the rings are transmitted through the column to the opposite rings, and consequently a far higher resistance is obtained than by the ordinary spiral winding round a central core.

Reinforced Concrete Staircase.—No. 7,489/11. *Mannesman. Accepted October 12/11.*—The inventor proposes that the flights of steps required in a staircase might be manufactured at a convenient spot and afterwards erected bodily in the building by attachment to the respective landings. Referring to other methods, he states that considerable difficulty has arisen when staircases have been built up from ready-made treads and risers—more particularly when the stringers have to be added separately. It is also stated that the system suggested possesses advantages in the case of spiral staircases. All bolts for securing the banisters and the staircase flights themselves are, of course, embedded in the concrete during the process of manufacture.

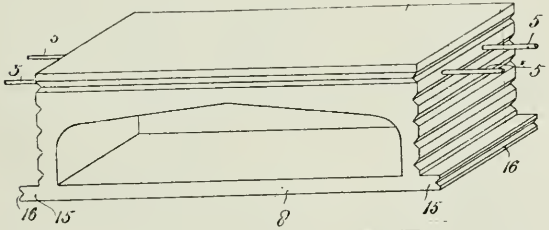




The Attachment of Shear Members.—*No* 23,435/10. *F. G. Engholm. Accepted November* 16/11.—The invention relates to another method of fixing steel stirrups to the main reinforcement so that they may be placed in position during erection to suit the requirements of the work. It is proposed to use small clips (*c*) bent to the contour of the bar (*a*) and adapted to receive the stirrups in the manner shown by the dotted lines in *Fig.* 1. When in this position the ends of the stirrups are bent upwards thus, with help of the clips, making an attachment by gripping the bars they encircle together. It will be readily understood that an arrangement of this nature enables the stirrups to be fixed on to more than one bar at once if it is deemed advisable.

Another Flooring System.—*No.* 11,444/11. *W. J. Swain. Accepted September* 28/11.—This is another endeavour to lessen the cost of erection for reinforced concrete floors by reducing the quantity of the centering and by doing away with the necessity of forming the floor in position. The floor designed by the inventor is made up of hollow-concrete blocks which may be cast where most convenient, and when completed appear as shown in *Fig.* 3. The tops of the blocks are smooth and finished, and therefore form the floor surface, whereas it is stated that, in other systems using similar blocks, only the core for the floor is made, and it is afterwards necessary to form a surface when the blocks are in position. Reference to *Fig.* 5 will show that the blocks are made with projecting flanges which come together and thus form boxes for the main and cross beams. It is claimed that the blocks may easily be placed in position and the beams subsequently formed between them, thus offering rapidity of construction and a homogeneous structure when completed.

Fig: 3



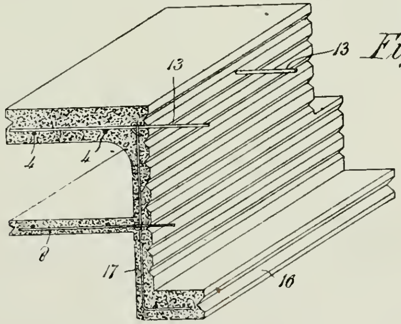


Fig: 4

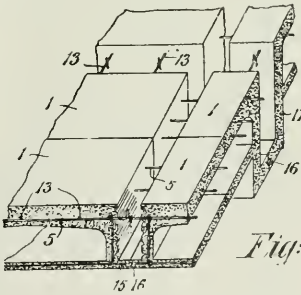


Fig: 5.

Fig. 1.

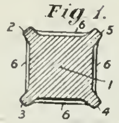
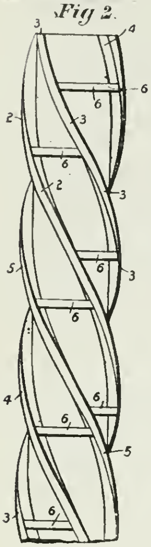


Fig. 2.



Another Steel Bar.—*No.* 27,373/10. *G. Hatton. Accepted November* 16/11.—The illustration shows another type of bar intended for use in steel reinforcement. In this instance the bar is rolled with longitudinal projecting ribs throughout its whole length and is afterwards twisted. The process transforms the straight ribs into spiral projections. At intervals the ribs may be connected by cross members (6). It is stated that bars of various sections may be manufactured on these lines, but the inventor prefers the square section shown. It is claimed that the bar ensures a much better anchorage in the concrete than can be obtained with other types.

A PRECIS OF PATENT SPECIFICATIONS.

Mixing Concrete.—*No.* 20,149/10. *John Wulstan Twinberrow.*—To save labour in mixing concrete for use, an apparatus is designed for attachment to a crane, so that the dry materials may be put in the hopper, and during the process of carrying them by the crane to the spot where the concrete is required, they are mixed with water and made ready for dumping. *a* is the hopper, pivoted at its bottom end to the bracket *n* on the crane jib. *b* is a door held by screw *c* and link *f*. A water-tank *g* with filler *h* is at the top of hopper, *i* being the communicating pipe with valve *j* to the bottom chamber. *p* are members so shaped that when skip is lowered to ground they always cause the door *b* to be uppermost. The dry materials in the proper proportions are filled *b*, water is put in at *g*. The crane then lifts the skip, and the trunnions at *o* are engaged. By raising and lowering the chain the materials are caused to roll from one end of the skip to the other, their passage being broken and mixture caused by the fast bars *k* across the interior. Water is then added, the operation continued, and the concrete is then ready for dumping in situ.

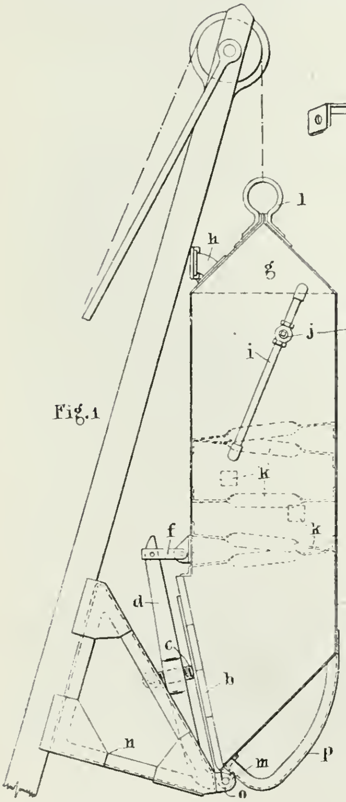


Fig.1

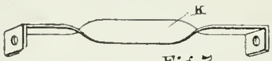


Fig.3.

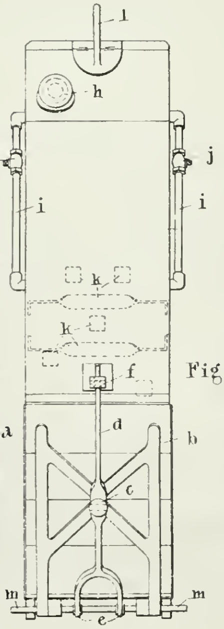


Fig.2

Concrete Piles.—*No.* 22,695/10. *Moritz Kahn.*—With the object of introducing a purely concrete pile into the ground, and yet being able to first drive a proper hole by ordinary driving methods, the inventor has designed a sharp-pointed nose to fit a hollow steel driving pile, which can be detached and brought to the surface without disturbing the hollow pile until this has been filled with concrete, when it can be withdrawn. This gives a concrete pile with a very blunt end as generally desired. Several alternative forms of construction are given. The simplest is a screwed point as *Figs.* 1 and 2. This is screwed into the lower end of the tube to be driven, and is extracted by means of a long box spanner lowered on to the square *e*. *Fig.* 11 shows the tube then left in the ground, and *Fig.* 12 the concrete pile left after filling the tube and then withdrawing it in the usual manner. *Figs.* 5 and 7 are alternative points which explain themselves.

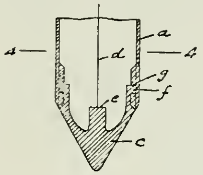


Fig. 3

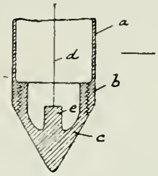


Fig. 1

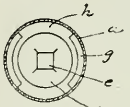


Fig. 4

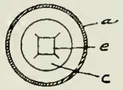


Fig. 2

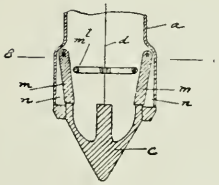


Fig. 7.

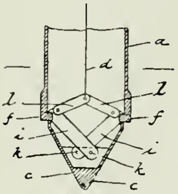


Fig. 5.

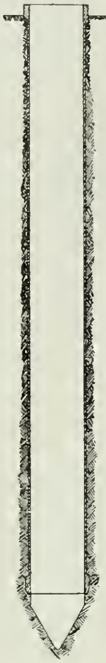


Fig 11

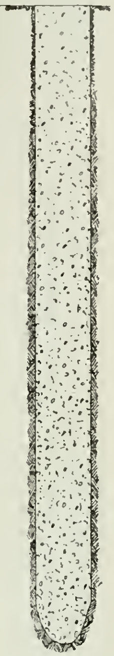


Fig. 12

Concrete Reinforcement.—*No.* 14,915/11. *Harry August Stockman.*—The inventor states that in lieu of employing rods or bars as reinforcement, singly or coupled together or in parallel with stirrups, etc., to take the shearing stresses, he employs two sets or series of parallel rods or bars in different planes forming tension and compression members and connected by inclined truss members, also employing corresponding transverse tension and compression members, the whole forming a mat, bolster, or skeleton wire structure of the required length and breadth; thus the tensile strength of reinforced concrete can, weight for weight, be increased above anything possible by the present methods or systems of reinforcing by the fact that the steel reinforcing medium is better distributed throughout the mass, and that every member is carrying its share or proportionate part of the total load. Hard-drawn steel wire is used, and is woven as *Figs.* 1 and 3. The main tensional members are double, as shown, and the connecting ties, etc., single strands. 1 and 2 represent two of the longitudinal wires of the one or upper set or series, and 3 and 4 two of

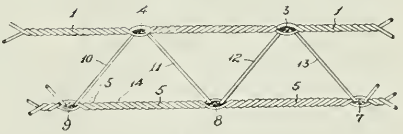


Fig. 3.

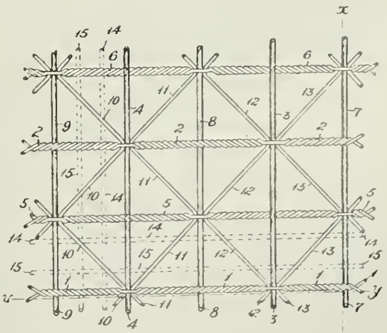


Fig. 1.

the transverse wires of the same set or series; 5 and 6 are two of the longitudinal wires of the other or lower set or series, and 7, 8, and 9 are three of the transverse wires of the lower set or series. Detailed descriptions are given in the specification as to methods and means of weaving and of various applications and forms of the device.

Concrete Railway Ties.—*No.* 21,341/11. *William Voorhees La Bau.*—The object of this invention is to "provide a railway tie composed of separate blocks, made in the manner of the voussoirs of an arch, whereby the concrete is placed in compression." The inventor states: "My invention consists in forming a railway tie of separate blocks or voussoirs, the contact surfaces of the central voussoirs being in planes which converge to a predetermined point above the tie, while the contact surfaces of the voussoirs adjacent to the central voussoirs are in planes which converge to a predetermined point below the tie, said last-named voussoirs serving as supports for the rails, and said tie being also provided with end voussoirs having contact surfaces which correspond with the contact surfaces of the adjacent rail supporting voussoirs, whereby the concrete of the tie is placed in compression and the greatest possible strength secured, the separate blocks or voussoirs being also bound together by longitudinal reinforcing rods which pass therethrough." *Fig.* 1 shows the tie or sleeper, *a a* being the central voussoirs, *b b* the rail carrying ones, and *c c* the end ones. Wood blocks *d d* are provided for spiking or fastening the rails, and all the voussoirs are reinforced with metal, similarly to *Figs.* 4 and 5, though the inventor does not confine himself to any set form of reinforcement.

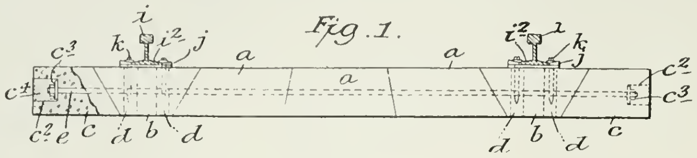


Fig. 1.

Fig. 4.

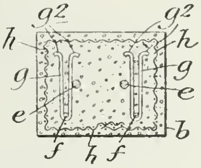
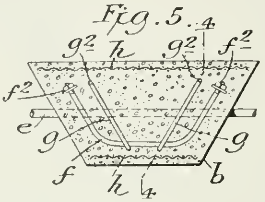


Fig. 5.



Column Reinforcement.—*No.* 11,513/11. *Lastra and Others. Accepted November* 30/11.—The invention consists of a new form of metal bracing specially adapted to hold the vertical reinforcing rods in position, as well as to facilitate the process of manufacture and render it possible to make such columns hollow. In the case of circular columns, as described by the inventors, the horizontal ties consist of a special spacing ring (*Fig.* 1) combined with internal and external locking rings (*Fig.* 2). When fitted together (*Fig.* 3) the perforations in the locking rings fit over the projections in the spacing ring. It is pointed out that such fittings combined with the vertical bars may be so built up that a rigid skeleton is obtained. When the columns are hollow and cast in moulds prior to erection the core may be filled with a wooden mandril, or other methods may be used to the same end

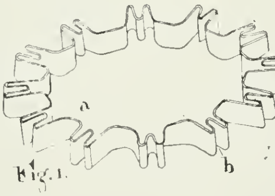


Fig. 1.

Fig. 2.

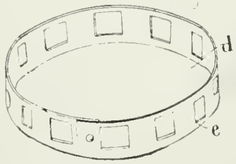
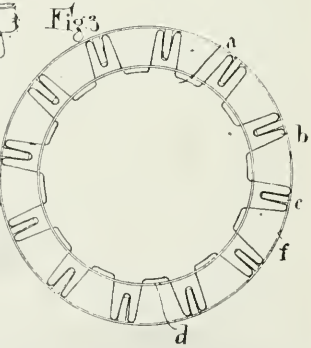
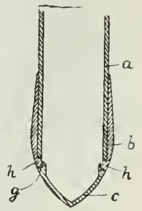
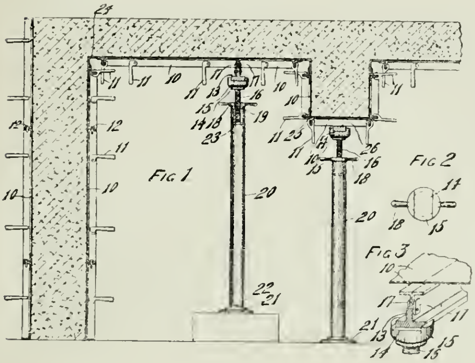


Fig. 3

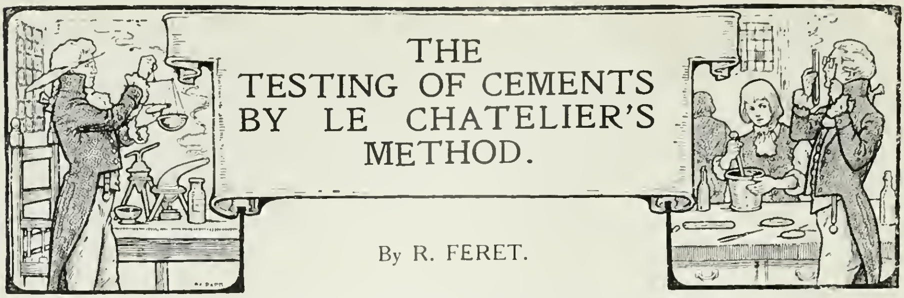


Concrete Piles.—*No.* 679/11. *F. W. Campbell. Accepted January* 10/12.—It has been found expedient in some cases to form concrete piles in the ground by simply piercing the hole and afterwards filling it in with concrete. The result is attained by first driving in a hollow preparatory pile to the desired depth and then withdrawing it. The method necessitates the adoption of a comparatively large driving point that must of necessity remain in the earth. Difficulties have also been encountered at the junction between the point and the tube, and it is suggested that a tapering collar (*b*) should be adopted here. The collar is attached to the tube, and on account of its increased diameter it clears a passage for the rest of the pile. During the process of withdrawal the collar presses back the surrounding soil, and thus prepares the ground for the reception of the concrete. Such a device does away with the large point which is required to prepare a free passage for the tube, and consequently a smaller point may be adopted and some economy obtained in the metal remaining at the bottoms of the openings.





Improved Centering.—*No.* 16,763. *Reichart Manufacturing Co. Accepted December* 14/11.—This invention is an attempt to reduce the expense of wooden moulds, etc., used in making concrete floors, beams, etc. The patentee proposes to use steel plates and adjustable supporting columns, which can be easily taken down and used over again. Referring to the drawing, 20 is a piece of pipe forming the body of the column, 21 is a loose base with a projecting piece 22, over which the pipe slips; 19 is a nut provided with turning lugs and machined to fit inside and on the end of the pipe; 16 is the screw engaging with the nut and carrying a head slotted to receive a rail or other supporting piece for holding up floors, etc.; the end of the screw which is inside the pipe is provided with a shoulder 23, which helps to steady the arrangement and strengthen the column. The method of using the steel plates and joining them with clamping hooks is clear from the drawing. The only point calling for remark is the special corner pieces provided and shown at 24 and 25.



THE TESTING OF CEMENTS BY LE CHATELIER'S METHOD.

By R. FERET.

An important communication was made by M. R. Feret to the French and Belgian Members of the International Association for Testing Materials at their meeting on January 20, 1912. The following is a somewhat shortened translation.—ED.

After having given an historical and analytical sketch of the way in which the Le Chatelier's boiling method has been adopted as the standard test for the soundness of cements by the International Association for Testing Materials, the author explains that objections have since been raised, chiefly by the German representatives, and a proposal will probably be made against its use at the coming Congress in New York.

The German manufacturers of Portland cement maintain that, in experiments made in different laboratories, the same cement has given expansions with the Le Chatelier test varying from 3·5 to 37 mm., whilst it has failed to condemn some cements which swell when tested by the usual German test, that of keeping pats of cement for four weeks in cold water. Objections have also been raised in other quarters, particularly by Mr. Butler before the Concrete Institute on March 17th, 1910.

The first objection brought forward relates to the lack of agreement of the results obtained. It must be remembered, however, that the various official prescriptions have not always been identical. The original rules, formulated by the French Commission for the Unification of Testing Methods on the basis of M. Le Chatelier's report, were as follows :—" The cylindrical test-pieces are to be 3 cm. diameter and 3 cm. high, moulded in metal cylinders 0·5 mm. thick, split along a generatrix and carrying, soldered at the two edges of the slit, two needles 15 cm. in length ; the increase of separation of the points of these needles is a measure of the expansion. The moulds, as soon as filled, are immersed in cold water. When completely set, and not more than twenty-four hours after setting, the temperature of the water is raised progressively to 100° in the course of from a quarter to half an hour. The temperature of 100° is kept constant for six consecutive hours, and the whole is cooled before making the final measurements."

These rules were first enforced in 1902 in the specification of the Ministry of Public Works, but modified in certain points. Thus the moulds were specified to be of brass, the preliminary hardening was to take place in moist air and not in cold water, and the period in boiling water was reduced to three hours. Lastly, whilst the ministerial instructions conserve the old rule concerning the time of preliminary hardening, the specification fixed a constant period of twenty-four hours for cements and forty-eight hours for limes. This difference was, however, only apparent, since the specification imposed a maximum final set of twelve hours for cements and thirty hours for limes, leaving an interval of less than twenty-four hours before applying heat. Further, each specification fixed a limit to the permissible expansion in each class of material.

The rules of the French military Corps of Engineers differ from those of the initial Commission for the Unification only in as much as they prescribe that, during the moulding and preliminary hardening, the two edges of the slit must be held together by means of a metal ring, 31 mm. wide.

The British standard specification fixes the hardening period at twenty-four hours, and gives further details as to filling the moulds. These are to be placed on a sheet of glass and filled with the gauged cement, whilst the two edges of the slit are gently held together. The mould is then covered with another sheet of glass, on which a small weight is placed, and it is immersed in cold water for twenty-four hours before making the hot test. The prescription is also made that the samples must be tested after twenty-four hours' and after seven days' aeration.

The rules proposed in Mr. Blount's report to the Copenhagen Congress are identical with the latter, except for an addition made—that anything used to hold the edges of the mould together during setting must be removed after twenty-four hours, before measuring the initial distance between the points of the needles. It was also agreed by the special Commission of the International Association to omit the permissible limits for cements in different states of aeration, since this Association has only to fix the methods of testing, leaving each authority free to indicate the state in which products are to be examined and the limits to be allowed. The French edition of the Congress report contains the further instruction that the ends of the needles are to be held in contact during the filling of the mould. This provision, which is undesirable, appears to be due simply to a mistranslation.

The principal differences between the specifications thus relate to the conditions of hardening between gauging and boiling. It is not clear why the French specification of 1902 provided for hardening in moist air, M. Le Chatelier having shown that immersion was preferable. The English recommendation to close the cylinder by glass plates, also advised by M. Le Chatelier, is excellent, and should be retained. The question as to the period of hardening needs closer examination. It would be inconvenient to make the time of hardening dependent on that of setting, which is itself very ill-defined; but, on the other hand, the imposition of a uniform time for all materials would favour those which are quick-setting as against those which set slowly, for it is found that, for the same mortar, the expansion is less the longer the time allowed for hardening. This explains the differences of expansion sometimes observed when the setting time is modified, for instance, by varying the fineness of grinding or the proportion of water used in gauging. It will probably be well to abandon the prescription of a fixed time, and to find some practical means of proportioning the time to the setting time of each sample. As to the various artifices proposed for hastening the hardening, they have the disadvantage of departing too widely from actual working conditions, and thus rendering the conclusions of the test suspect.

It should not be difficult to reach an agreement as to whether the hot test should last three or six hours. The British provision with regard to aeration introduces a source of error, as the aeration might take place under widely different conditions. Agreement has already been reached as to the other factors affecting the result.

There are certain errors, however, which are to be attributed entirely to the want of care or reflection of the operators, and which cannot be provided for or stated in the specifications. Thus it sometimes happens that, owing to the excessive curvature of the brass sheet forming the mould, the two edges of the slit press on one another; some operators even arrange this intentionally in order to prevent the escape of mortar from the slit. Evidently a certain amount of expansion must take place before this pressure is relieved, and the final expansion measured is less than it should be. Also it may happen that the needles touch and press on one another at some point, with similar results. It is evident that this test, like all others, must be performed intelligently.

Some laboratories, on the other hand, employ insufficiently curved moulds, the edges of which have to be held together during filling. In the course of the test the mould may part from the test-piece and so cause an increased separation of the needles. This separation may also occur during cooling, on account of the unequal contraction of brass and cement, and can then be detected by measurements before and after cooling.

The moulds should preferably have such a curvature that the edges of the slit are just in contact, or, better, exert a slight pressure which will just be overcome by the weight of the mortar and of the weighted plate placed on it. It seems preferable not to fasten the moulds in any way. The escape of a little mortar from the slit is immaterial, as such ridges are easily removed after hardening.

Another error lies in the mode of fixing the needles to the cylinder. Most manufacturers bend the lower ends of the needles into arcs, which are soldered to the cylinder in the plane of the median section, thus forming a stiffening rib. This is avoided if the bent ends are soldered along the edges of the slit. Table I. contains some experiments to determine the influence of such stiffening ribs, and it also shows the influence of the time of boiling.

TABLE I.

A. Three similar moulds, having the bases of the needles soldered along the edges of the slit.
B. Mould with the needles bent and soldered in the median plane. forming two arcs 15 mm. long.
C. Mould with the soldered arcs 30 mm. long.
C^1. Mould similar to the last, except that the thickness of brass is 1 mm. instead of ½ mm., as in the others.

Material.			Time of Boiling in Hours.	Expansion.			
				Moulds A.	Mould B.	Mould C.	Mould C^1.
Portland Cement			5	2, 3, 3·5	2·5	2	1
Underburnt Cement			2	61, 65, 69	58	59	35
Hydraulic Lime			6	3·5, 4·5, 5	3	3	1
Roman Cement...			2	25	23	21·5	10
	Cement.	Lime.					
Mixtures of a good Portland Cement and a bad Hydraulic Lime	95	5	6	3, 3, 3	3	3	1
	90	10	1	7, 7, 8	6·5	6·5	4
			2	10, 11, 12	9·5	9	5
			5	10, 11, 12	10	9	5·5
	85	15	½	14, 15, 16	13	14	7
			1	24, 24, 24	21	21	10
	75	25	½	28, 29, 33	30	22	12

It will be seen that the influence of the ribs is distinct, although not large. The thicker mould gives considerably lower results. The expansion of the 90 : 10 mixture is the same after five hours' as after two hours' boiling, which would suggest that the test period of three hours is better than six hours, but the results would probably be different with a cement the expansion of which was due to magnesia.

Interesting experiments, shown in Table II., were made by Mr. Butler, using one and the same cement kept out of contact with air. Moulds of various kinds were used, and the experiments were repeated, at intervals of two days, by three operators, two of whom had made thousands of tests in the course of some years, whilst the third was a novice.

TABLE II.

Number of Moulds.	Nature of Moulds.	Expansion.			Separation of Needles in mm. under a weight of 2 lb.
		Operator 1.	Operator 2.	Operator 3.	
1	New Nickel Mould	5	2·5	2·5	2
5	New Brass Moulds of different makes	6 – 7·5	4 – 5·5	4 – 5·5	11·5 – 13·5
6	Used Brass Moulds	8·5 – 11	4 – 6	6 – 7·5	20 – 27
1	Special Mould of Thin Brass ...	8	3	8·5	>40

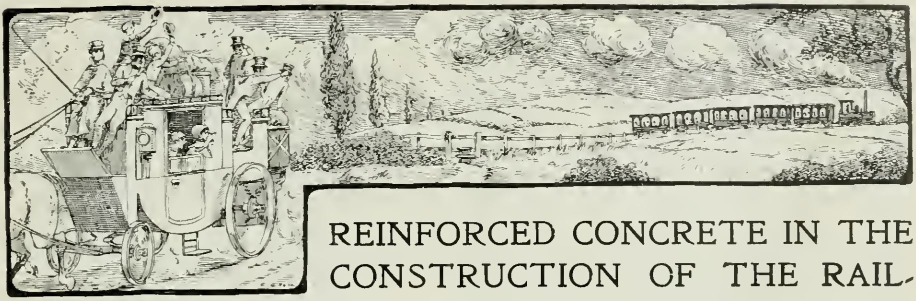
The expansions obtained by the first operator are much greater than those obtained by the others, but the tests were made two days earlier, and the cement may have lost some of its expansive power, although stored in a closed vessel. They should have been checked by tests at a later period. As for the second and third operators, their agreement, at least with new moulds, is excellent, in spite of their difference in skill. Taking the expansions observed by any one operator, it will be seen that they follow an increasing order as arranged in the table. Mr. Butler measured the flexibility of the moulds by threading them on a sleeve moving on a horizontal rod, so that the diameter passing through the slit is horizontal; a load is then applied by means of a stirrup, also contained in a loose sleeve. The results show that the expansions observed are always greatest in the most flexible moulds.

M. Heyn has expressed the opinion that unequal internal stresses may arise during the working of the metal, and may alter after many tests, especially in contact with boiling water containing lime. This would account for the differences in flexibility between old and new moulds. In any case, the flexibility of the mould influences the results in a way that cannot be exactly defined, and it would perhaps be well to specify that, in cases of dispute, tests should be made with moulds stamped by an official laboratory as the result of flexibility tests.

The elasticity of the mould and the time of hardening are, then, the two principal conditions which need to be better defined in the official rules for the test, but it is unlikely that the enormous deviations found in some experiments are to be attributed only to these factors. They are more probably due to differences in the initial state of the cements and the processes of aeration to which they have been subjected, or to carelessness of the kind just alluded to, which, although easily avoided by care and reflection, is quite natural on the part of workers untrained in tests of the kind. We cannot expect to obtain absolutely concordant results, any more than in other tests; but the extreme simplicity of the process suggests that it should be possible, by fixing the manner of operation very precisely, to reduce the errors of experiment to a minimum.

The author intends to make a further communication relative to the extent to which the indications of the Le Chatelier test really correspond with real differences in the properties of hydraulic products.

In a communication presented at the same meeting, M. Bied has shown that an unsound cement may lose its expansive properties even when stored in a sealed case. The German samples, although sent out in sealed boxes, may therefore have undergone changes, which may account for the divergences observed.

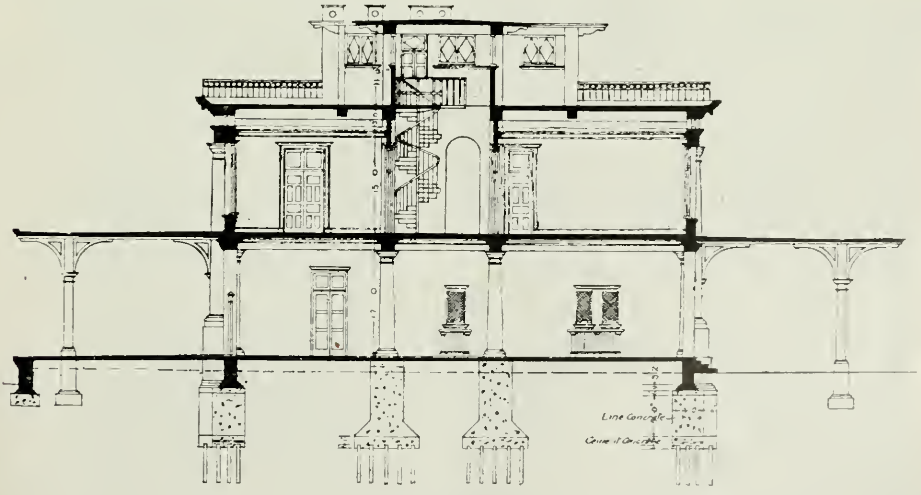


REINFORCED CONCRETE IN THE CONSTRUCTION OF THE RAILWAY STATION, CANTON, CHINA.

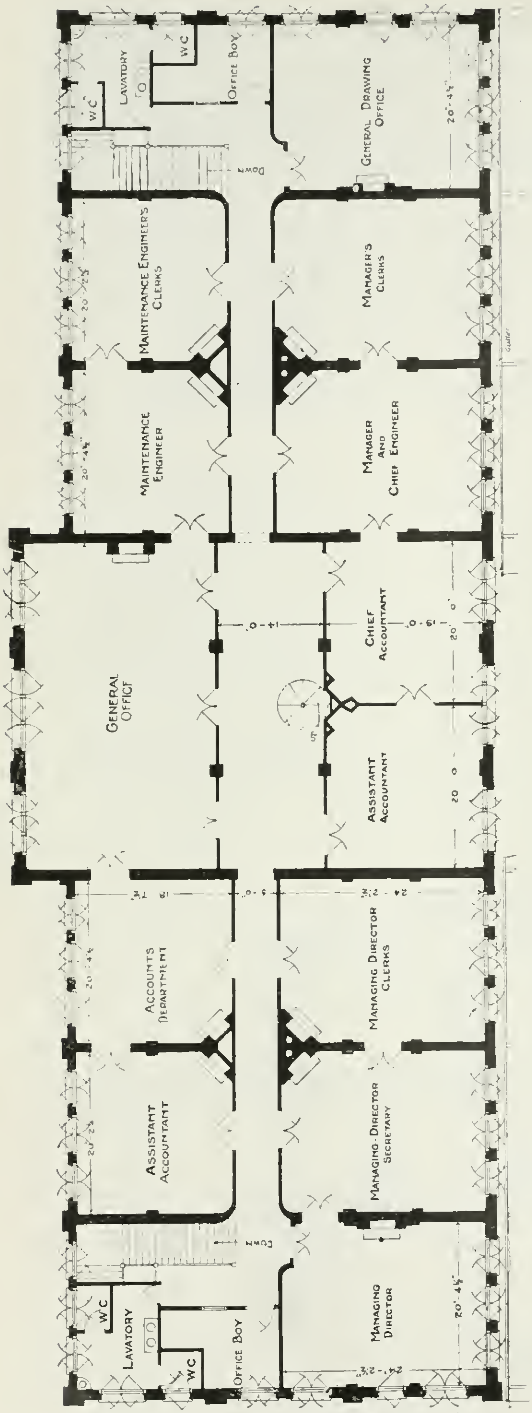
The following account of the advance which reinforced concrete is making in the Far East will be particularly interesting at the present moment, and we fully expect that this method of construction will increase in favour with architects and engineers in China.—ED.

CANTON, China, possesses a modern, well-built and fully equipped railway station, built of reinforced concrete, which reflects much credit upon the Chinese Managing Director of the Canton-Kowloon Railway, Mr. Wei Han, who in 1909 commissioned Messrs. Purnell & Paget, architects and engineers, of Canton, China, to carry out the design and construction of a building in these materials, which was a radical departure in Chinese methods of construction. The foundation-stone for this building was laid on April 7th, 1909, in the presence of a distinguished gathering of representative foreign and Chinese officials from Canton and Hong-Kong.

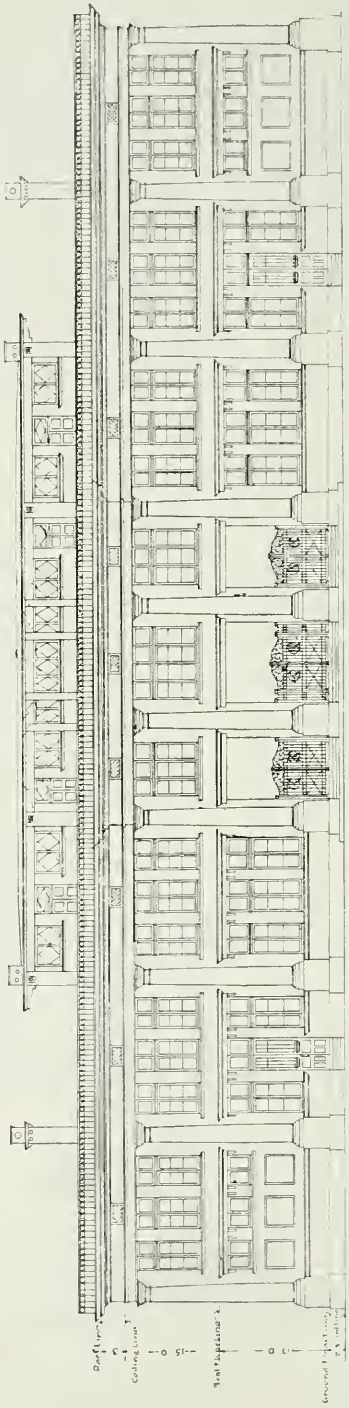
The main portion of the building, two storeys in height, is 172 ft. long and 54 ft. wide, the height of the main cornice being 38 ft. from the ground. Service rooms on the roof give a total height of 48 ft.



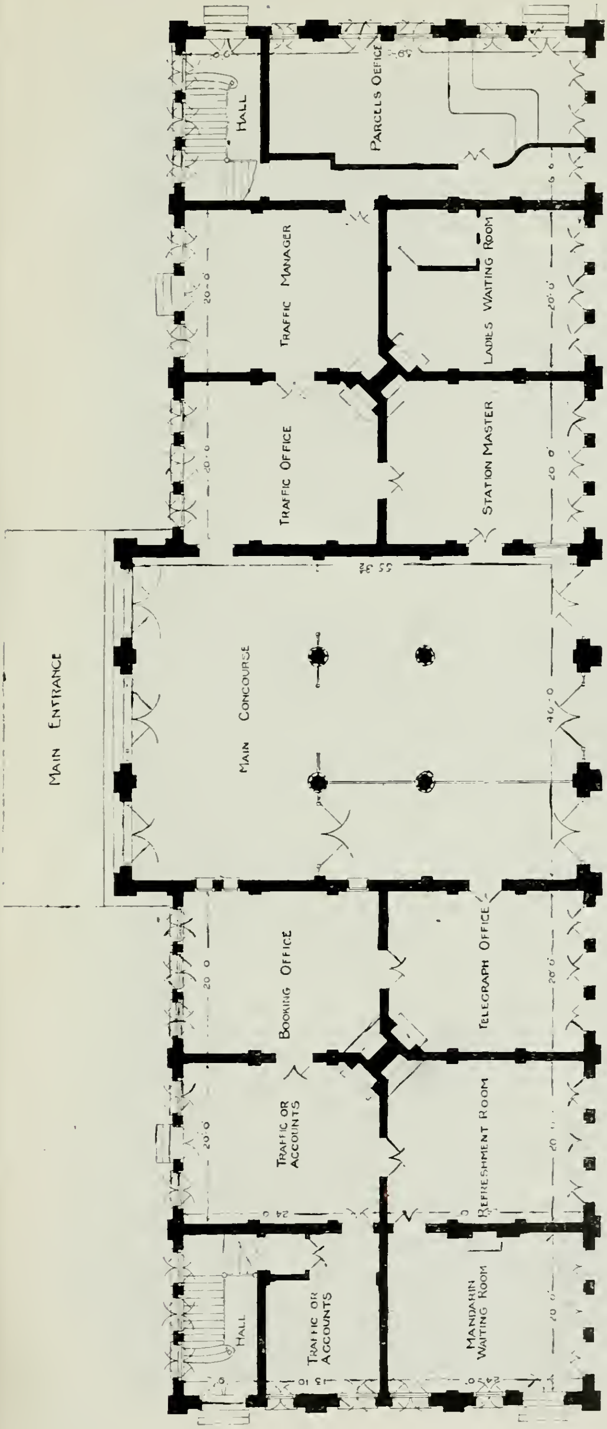
RAILWAY STATION, CANTON, CHINA.



First Floor Plan.



North Elevation.
Railway Station, Canton, China.



Ground Plan.
Railway Station, Canton, China.

On the ground floor the central feature is the main concourse, 40 ft. wide, giving access to a spacious platform leading to the train shed. The booking and telegraph offices are arranged on one side of the concourse, while the Station Master's office is accessible from the other side.

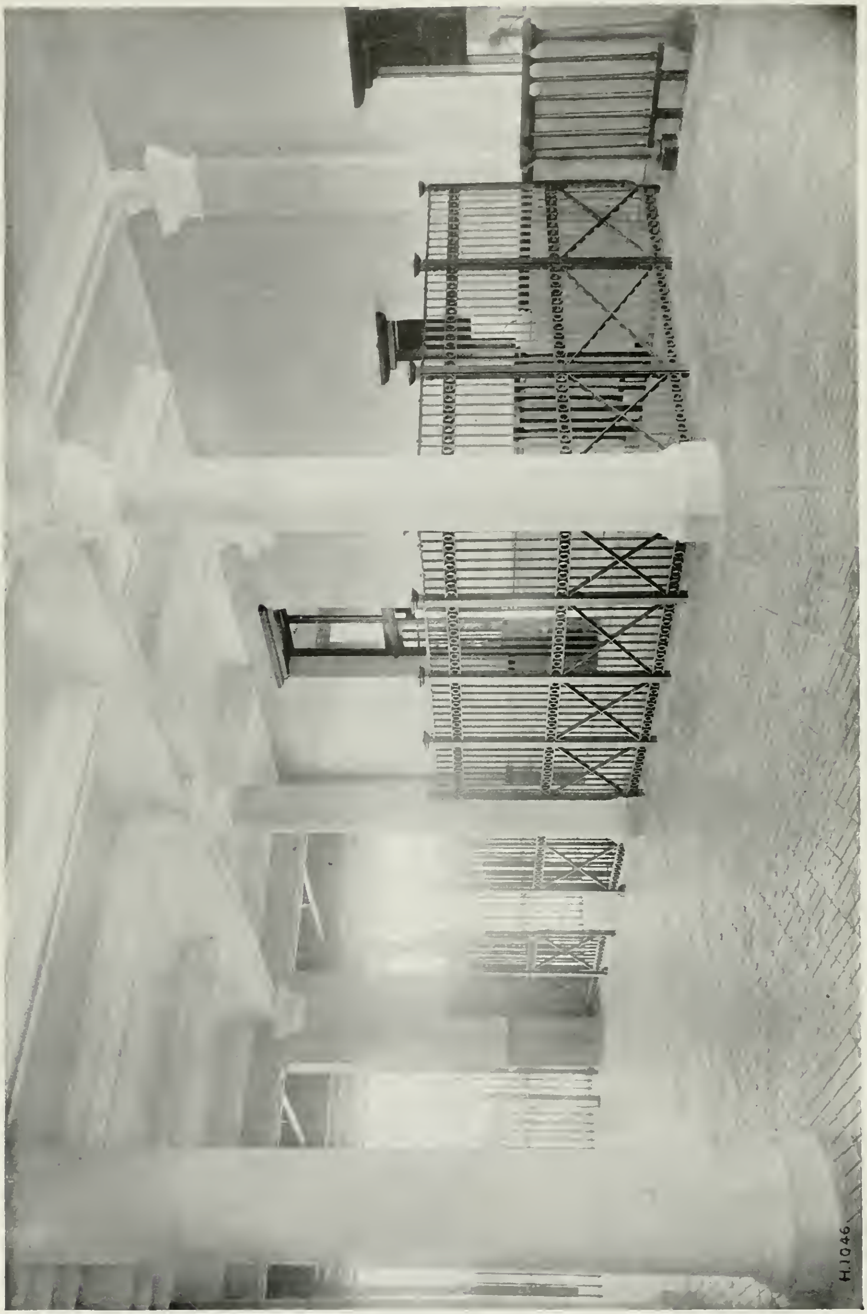
The ground floor also contains accommodation for Traffic Department, Parcels' Offices, Refreshment Rooms and Waiting Rooms for first, second, and third classes, women and officials.

The first floor is allotted to the various departments of the railway. The Chinese Director's room, the Engineer-in-Chief, with his staff of assistant engineers and draughtsmen, and the Accountant's departments occupy this entire floor.

The service rooms are on the roof, with the dining room, lavatories, kitchen, etc., for the staff.

The building is thoroughly fire-proof, being constructed in brick and concrete. The walls are built of local brick, the exterior being faced with red pressed brick from the new

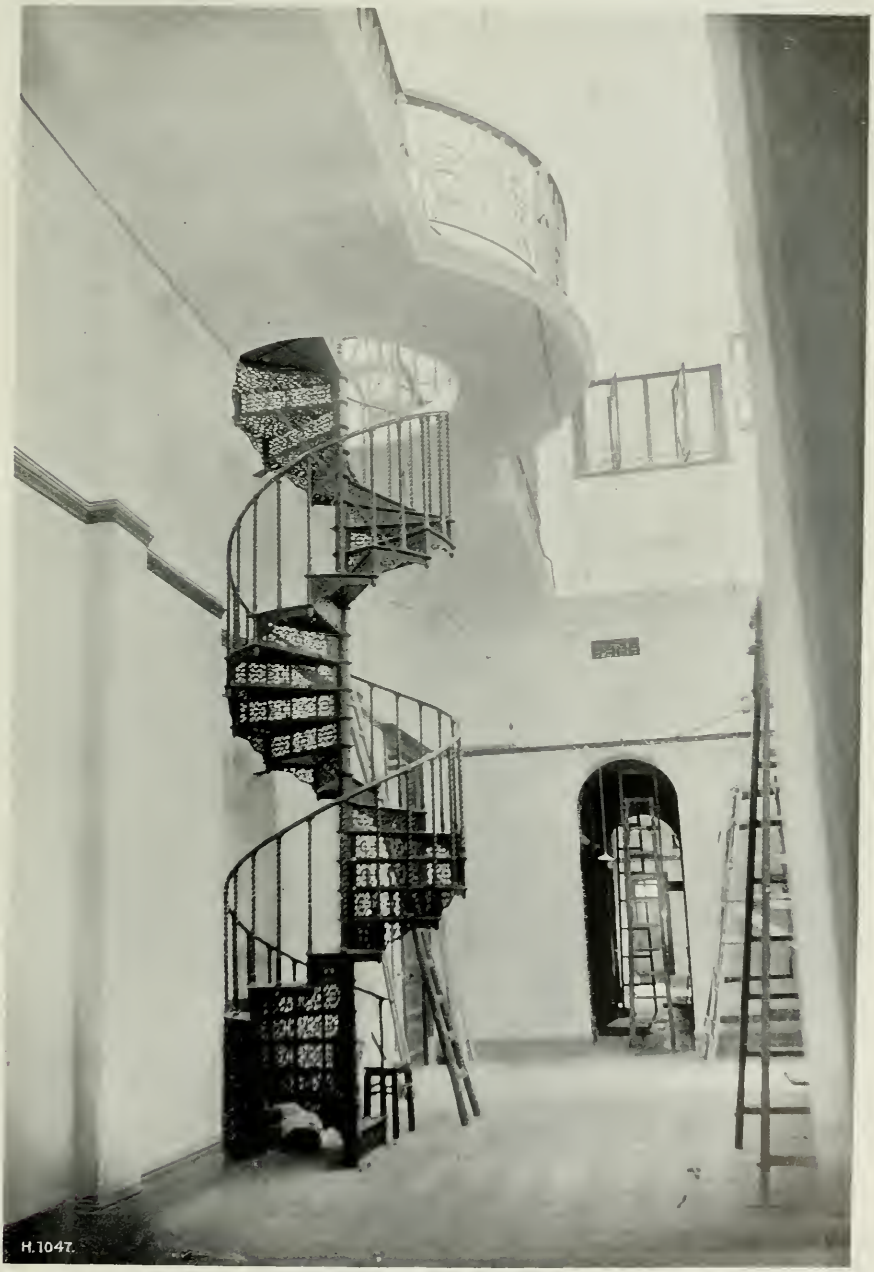
Kwong Tung Brickworks. All floors and roofs, with beams, girders, and columns, are of reinforced concrete.



Main Concourse, Ground Floor.

Railway Station, Canton, China.

The two main staircases are also of reinforced concrete, with local ornamental wrought-iron balustrading and tiled treads. The central stair leading to the roof is of wrought iron. The joinery and interior wood finish is entirely of Burmese teak.



Staircase and Cantilever Gallery.
Railway Station, Canton, China.

All floors are tiled throughout with cement tile, except the lavatories and entrance halls, which are of imported tesselated tile. The concourse and train platform are tiled with ribbed vitrified tile of local manufacture. The results obtained for the traffic for which these floors are made is particularly satisfactory. These tiles afford a hard, smooth-wearing surface, durable, easily kept clean, and affording an excellent purchase for the feet.

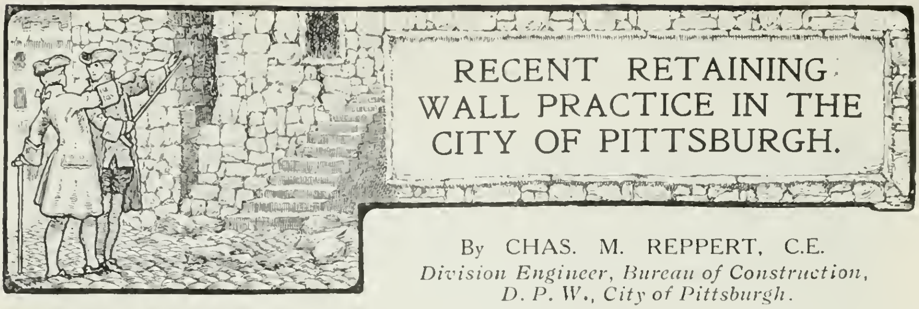
Service Rooms on Roof.
Railway Station, Canton, China.

The design of the façade is the Roman doric order, which is in keeping with the distinctive purpose of an important railway terminus.

The design and execution of the work was done under the direct supervision of the architects, Messrs. Purnell & Paget. Credit is due to Messrs. Frank Grove, M.Inst.C.E., Engineer-in-Chief of the Canton-Kowloon Railway, and W. G. Moore, District Engineer, Canton-Kowloon Railway, for the successful results of the architects' design. Mr. R. Claude G. Ogilby, representing the architects, was in direct charge of the work.

The building has been in use since the opening of the first section on December 5th, 1910, and more fully since the formal opening of the entire line —some ninety miles—from Canton to Hong-Kong on October 4th, 1911.

The Canton-Kowloon Railway will eventually form a connecting link with the Canton-Hankow Railway, now in course of construction, which upon completion will give through traffic from Hong-Kong to Paris, *via* Peking and the Trans-Siberian Railway.



RECENT RETAINING WALL PRACTICE IN THE CITY OF PITTSBURGH.

By CHAS. M. REPPERT, C.E.
Division Engineer, Bureau of Construction, D. P. W., City of Pittsburgh.

The following particulars of recent retaining wall practice in the City of Pittsburgh, U.S.A., were presented before the Structural Section of The Engineers' Society of Western Pennsylvania and were included in that Society's Proceedings.—ED.

Introduction.—It is proposed to describe some of the retaining walls recently constructed by the Department of Public Works, Bureau of Construction, of the City of Pittsburgh, to review the methods used in their design and the assumptions upon which the designs were based. Until recently it has been the practice to construct these walls of masonry and plain concrete. Such walls may be described as gravity walls to distinguish them from the reinforced concrete walls whose strength to a much greater extent depends upon the internal strength of the structure.

During recent years the construction of walls of reinforced concrete has been resorted to. The stability of walls of this character depends upon the weight of the masonry, the weight of the superimposed earth resting on the back and heel of the wall, and the internal strength of the structure. In some designs the frictional resistance of the earth prism resting on the heel of the wall has been introduced as a force resisting overturning. Since, under certain conditions, the wall may fail without bringing this force into action, the writer is of the opinion that sufficient reliance cannot be placed on its action, and the frictional resistance has therefore been disregarded.

Comparing this new type of wall with the old gravity type, the quantity of concrete is much less, but the cost per unit is greater. The foundations of the reinforced walls need not be carried to so great a depth since the foundations can be spread and the unit base pressure reduced. In municipal practice in this City, however, it is not possible, ordinarily, to extend the toe beyond the face of the wall since the structure must be within the street lines, and for this reason the economical design of walls is limited.

Further comparing the reinforced type of wall with a gravity wall and assuming that in each instance the resultant of the pressure cuts the middle third, the ordinary requirement for the gravity wall, the reinforced wall will possess a much greater factor of safety on account of its ability to resist internal strains and consequently to resist pressures that cause the resultant to cut the base outside of the middle third.

METHOD OF DESIGN, EARTH PRESSURE.

Notation.—The notation and formulæ for earth pressure used in this paper are given in the following list. (See *Figs.* 1 and 2.)

G = Weight of wall and earth.
h = Height of wall,
b = Base,
P = Normal thrust $= \frac{h^2\gamma}{2} \; \frac{1 - \sin\phi}{1 + \sin}$
ϕ = Angle of repose,

γ = Weight of earth filling,
p = Equivalent fluid pressure,
P = Normal thrust $= \frac{1}{2}ph^2$
M_r = Moment of resistance to overturning for wall and superimposed earth = Ga
M_t = Overturning moment of thrust for Normal thrust $M_t = \frac{1}{6}ph^3$
M_s = Moment of stability of wall $= M_r - M_t$,
C = Compression at toe.
Q' = Thrust developed by tipping of earth wedge $= \frac{K\gamma}{2}h$
Q = Thrust developed by sliding of earth wedge $= \frac{\pi}{8}h^2\gamma$
U' = Thrust necessary for overturning $\frac{1}{3}$ h from base.
U = Thrust necessary for overturning $\frac{2}{3}$ h from base.

The formula used for the determination of the lateral thrusts is in the form $P = \frac{1}{2}h^2\frac{1 - \sin\phi}{1 + \sin\phi}$, in which P is the total normal lateral thrust, γ is the weight of the material per unit, h is the height, and ϕ is the angle of repose for the material. This formula, which is commonly known as Rankines formula, is based on the assumption that the earth is a granular mass without cohesion and applies to walls where the surface of the filling behind the wall is horizontal. The lateral pressure thus developed is termed the normal thrust to distinguish the pressures determined in this way from those determined by the other formulæ applied for different loadings. The point of application of the normal thrust is one-third the height of the wall from the base, and only the horizontal component of the resultant is considered, the friction being neglected.

Briefly reviewing the subject of earth pressures it is obvious that the pressures to be provided for and their points of application will vary with the character of the materials and with the conditions existing. Where possible, test pits should be sunk, particularly where the wall is to retain an old fill. The character of the fill, however, is seldom definitely known in advance of the construction, so that too much reliance should not be placed in any formula. Where walls are constructed to retain clay fills, placed on inclined or impervious slopes, it would appear to be good practice to make provision for possible slipping or tendency of the fill to slip. The writer believes

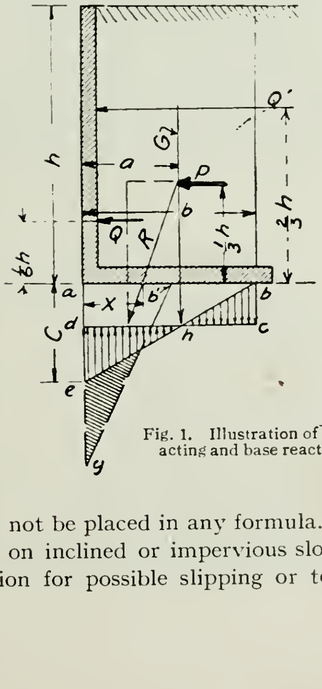


Fig. 1. Illustration of forces acting and base reactions.

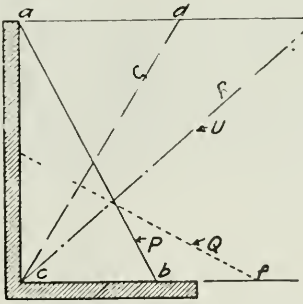


Fig. 2. Graphic illustration of earth pressures.

Design of Retaining Walls.

that as the fill settles and becomes compacted that these pressures will decrease, hence high unit stresses have been allowed under some conditions. Under what may be considered the working loads the stresses have been held at fairly low values to provide for possible increases in the loading, and on account of the fact that this type of construction is comparatively new.

DESIGN OF REINFORCED CONCRETE WALLS.

In designing reinforced walls it is frequently of considerable assistance to compare the resistance of the proposed design with a gravity wall of given base in terms of the height. This comparison is particularly valuable when gravity walls have been built in the locality where it is proposed to construct the reinforced wall. Having decided upon the value of the equivalent fluid pressure under the thrusts developed by which the resultant is to cut the middle third of the wall, the overturning moment may be equated about the toe of the wall with the approximate weight of the proposed wall, the width of base being variable. In this manner the width of base of a reinforced wall can be determined, having the same resistance under the normal thrust as the gravity wall with which it is compared.

Under loads exceeding the normal thrust the reinforced wall will have a much greater factor of safety owing to the ability of the structure to take tension. The stresses in the steel can be proportioned according to conditions so as to withstand much greater thrusts than could be resisted by the gravity wall. The stress in the steel is fixed so as to be at ordinary working values under the normal loading, with higher stresses under the other and more excessive loadings. If the full strength of the wall up to or approaching the point of overturning is to be developed then the stress in the steel at this point should not exceed the elastic limit.

In the upper portion of the wall the stress in the steel is generally kept at a lower value under the normal pressure so as to make provision for slips, the thrusts produced by the tipping of the fill and possible pressure produced by freezing.

In comparing the strength and stability of gravity walls with reinforced walls, it is necessary to consider their relative strength under different conditions. In the gravity walls, when the resultant cuts outside the middle third tension exists in the back of the wall and the stability of the wall will then depend upon the character of its construction. Any considerable amount of tension, however, would, of course, make the wall unsafe. Therefore, as a unit the reinforced wall on account of its ability to take tension gives a structure having a much higher factor of safety.

The gravity wall is better able to resist excessive pressures applied to only a portion of the wall, than a reinforced wall designed to resist normal pressures only, on account of the nature of its construction admitting of a better distribution of pressures. When the resistance of a gravity wall is exceeded by the thrusts, the wall in general falls slowly, while the reinforced structure would be more likely to fall suddenly, probably resulting in much greater damage. It is therefore essential in designing reinforced concrete walls to keep the stresses sufficiently low so as to provide for possible loads exceeding the normal pressure. However, under these excessive loads, such as are computed in accordance with the formulæ for Q and Q^1, the stresses may be at a much higher value.

The line of action of the various forces and resultant pressures at the base and floor reaction are shown diagrammatically in *Fig.* 1. In this diagram the rectangle *abcd* represents the uniform weight of the wall and superimposed earth. The pressure triangle *abe* represents the upward pressure on the base of the wall under the normal thrust p. The triangle *hbc* represents the resultant downward pressures on the floor, and the triangle *dhe* represents the resultant upward pressures on the floor. The reaction at the base under the thrusts Q^1 is indicated diagramatically by the triangle *ab'g*.

Fig. 2 shows diagrammatically the thrusts applied to the front wall under the different loadings. By the use of a diagram similar to this, the loads being plotted in lb. per sq. ft., the strength of the front wall and ribs may be proportioned to withstand the conditions imposed.

The following is the data used in designing the reinforced concrete :

Modulus of elasticity of steel, 30,000,000.

Modulus of elasticity of concrete, 2,000,000.

Ratio 15. Anchorage 40 to 50 diameters.

Shear without diagonal reinforcement not to exceed 40 lb. per sq. in.

Computations for reinforced concrete sections by straight line formula. Steel in direct tension 20,000 lb. per sq. in. Steel reinforcement used in the designs was square twisted rods of medium steel. It was bent cold to the required shape shown on the contract plans.

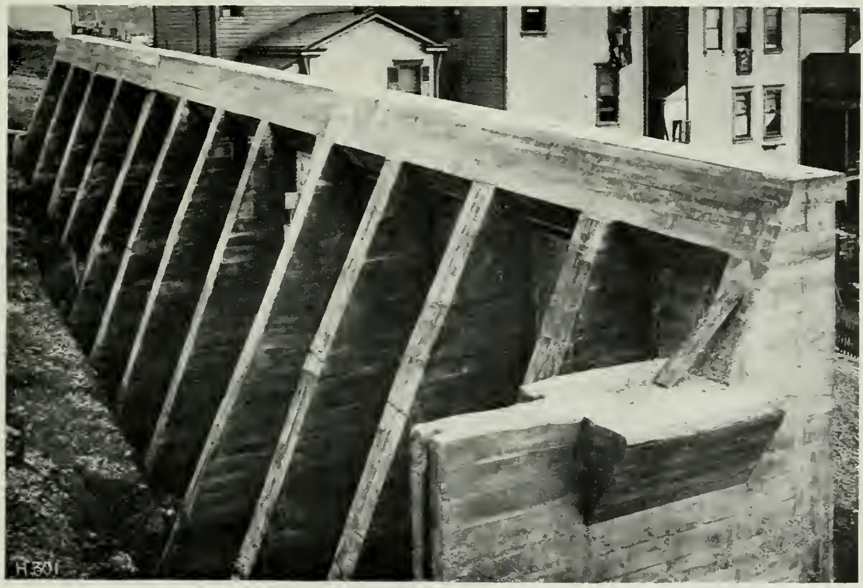


Fig. 3. Retaining Wall, Dilworth Street, Pittsburgh.

DESIGN OF DILWORTH STREET RETAINING WALL.

The Dilworth Street wall, a portion of which is shown in *Fig.* 3, was erected in connection with the grading, paving, and curbing of Dilworth Street. It was erected during the summer of 1909. In preparing plans for this wall comparative designs were made to determine the most economical type of wall. The site upon which the wall is built is a steep inclined hillside, upon which an old fill about 8 ft. to 10 ft. in depth had been placed. This old fill was of conglomerate material, composed of clay, loam, and also contained a large quantity of rubbish.

Test pits were excavated showing that shale of good quality was to be found at a depth of about 9½ ft. below the surface at the deepest section. The character of the material overlying the shale was such that it was deemed unsafe to construct a plain gravity wall without carrying the footing to the shale. The design of the gravity walls for comparison was therefore based upon carrying the footing to rock. The lateral thrusts, however, were computed for a depth of 33 ft. for the deepest section,

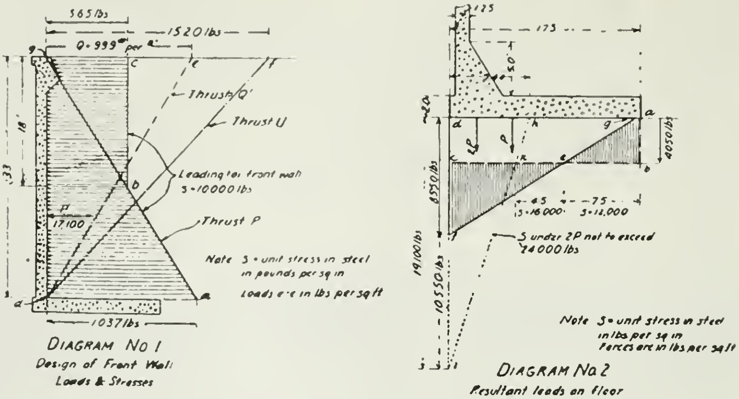


DIAGRAM No 1
Design of Front Wall
Loads & Stresses

DIAGRAM No 2
Resultant loads on floor

Fig. 4. Details of design.
RETAINING WALL, DILWORTH STREET, PITTSBURGH.

which section was compared with the cost of constructing a reinforced retaining wall 35 ft. in depth at the same point ; for the other depths the comparison and designs were made in a similar manner.

In deciding upon the proper co-efficient for computing the lateral thrusts to be resisted by the proposed wall, there was taken into consideration the fact that the clay filling would be placed upon a steeply inclined surface and underlaid with several feet of fill of poor character and that this material was underlaid by sloping impervious rock strata. It was therefore deemed advisable that the wall be constructed to resist a fill whose angle of repose ϕ would approximate 33 degrees 42 min. Further, on account of the nature of the material underlying the fill, it was considered necessary to provide an additional factor of safety for loads in excess of the normal computed loads. These excess loads might come from a slipping of the earth wedge of the new fill or from the tipping over of this wedge.

Design of Reinforced Concrete Retaining Wall for Height of 35 ft.—The reinforced retaining wall finally adopted, a section of which is shown in *Fig.* 4, was designed so that it would have a resisting value under normal thrust P equivalent to that of the plain wall with a base of $\cdot 5h$, and at the same time be safe under more excessive loads. The wall was, therefore, designed so that the resultant due to the lateral thrust produced by the fill under the assumption $\phi = 33$ degrees 42 min. would pass through the middle third and the stresses in the structure to be at working value. Under higher loads, such as might be caused by forces Q or Q^1, or pressures developed by freezing or settlement of fill, the stresses allowed were higher, approaching the elastic limit for maximum possible conditions.

The width of base was determined in accordance with the method stated before under design of reinforced concrete retaining walls. The required width of base as computed was 17·2 ft., and the width taken for the design was 17·5 ft., including a 6-in. toe projection.

The conclusions drawn from the comparative estimates made for the 30 ft. and 21·5 ft. heights indicate that the reinforced concrete walls would be as economical to construct as a gravity wall with a base ·4 the height, but that the reinforced concrete type would possess a far greater factor of safety. It was therefore decided to construct the entire wall of reinforced concrete.

The reinforced wall, as before stated, was designed for three depths, 35, 30 and 21·5 ft. respectively from top of wall to bottom of floor. The wall was also designed so as to rest upon piers extended along the front of the wall and underneath each rib. The data used in the design conforms to that given under design of reinforced concrete retaining walls. In this discussion only the design of the deepest section is given, the design of the other sections being made in a similar manner.

Design of Front Wall.—The front wall transmits the lateral thrusts to the ribs, and is designed as a slab fixed at the ribs. The connection between the ribs and the front wall was designed so as to fix the wall to the ribs, and as placed this assumption is practically correct. The bending moments at the centre of the slab were computed in accordance with the formula $wl^2 \div 15$ and at supports $Me = wl^2 \div 12$. In actual detailing the percentage of steel is varied so as to give convenient spacing to facilitate erection.

Diagram No. 1, *Fig.* 4, shows the triangles of pressure applied to the front wall, the loadings marked thereon being in lb. per sq. ft.

The front wall slabs were designed as follows :

The stress in the steel is equal to 10,000 lb. per sq. in. under normal loading from the base to the point b, which is 18 ft. below the top of the wall. From this point up the wall was considered to be under a uniform lateral thrust of 565 lb. per sq. ft., with a tension in the steel of 10,000 lb. It will be seen from an inspection of this diagram that under the load Q' the steel at the top of the wall will be stressed to about 18,000 lb., while at overturning the steel would be stressed to 27,000 lb. per sq. in. The unit stresses used are low for reinforced concrete work, but in fixing these stresses the character of the construction was taken into account.

In connection with the detailing of this wall it will be noticed at the ribs the rods are placed through holes punched in structural steel angles. This assures the alignment and securing of the rods, and at the same time affords a secure manner of tying the front wall to the ribs. A double line of reinforcement is used in the front wall. Some difficulty was encountered in placing the rods in the back row, and it would probably be more economical to design the front wall as a simple slab with bent rods for the negative moment at the points of support.

Design of Rib Reinforcement.—The ribs were computed as cantilevers fixed at the base, where they are tied into the floor, which is held by the superimposed load of earth. Tentative designs were made for panel spacing of 7·5 and 9 ft. The 9-ft. spacing proved to be the more economical.

The rib rods are run down the back of the wall, some of the rods extending into the projection at the heel, and some of them being placed under transverse 1-in. rods, which in turn engage the reinforcement in the floor. In this manner the secure anchorage of the rib rods is obtained and the distribution of the stresses between the floor and the rib effected. To insure the bond between the floor and the ribs vertical ties were also added.

The method of designing the reinforcement of the ribs was similar to that employed for designing the front wall. The ribs were investigated in sections 5 ft. apart, the percentage of steel determined, and the rods were carried a sufficient distance beyond the point required in order to give secure anchorage, then were bent as shown in order to avoid crushing the concrete.

The stress in the rib reinforcement varies from 10,000 lb. for the normal loading up to 40,000 lb. under the ultimate load, and was proportioned to develop the same strength as that of the front wall. Stress diagrams for the different sections and under the different loads were drawn similar in form to those prepared for the front wall.

Design of Floor.—Diagram No. 2, *Fig.* 4, illustrates the method used in fixing the unit stresses in the floor reinforcement. The floor reinforcement consists of double reinforcement so as to provide for negative moments at the base of the ribs and so as to carry the resultant pressures.

Under the normal thrust *P* the resultant cuts nearly at the middle third, so that the upward reaction is a triangle, the maximum pressure being twice the average at the toe. This condition is shown graphically in Diagram 2, *Fig.* 4, being indicated by the pressure triangle *gdf*. The downward pressure caused by the weight of superimposed earth is taken as the weight of the wall *plus* the earth and considered as uniformly distributed (see Diagram 2, area *abcd*). The difference between the upward and downward pressure gives the resultant pressure which the floor must be designed to carry. For this condition the resultant pressures are shaded in Diagram 2.

As the loading is increased beyond that produced by the normal thrusts a greater width of the floor is subjected to the downward pressure; at the same time the intensity of upward resultant at the toe increases. In order to proportion the wall so as to give all parts equal strength under the same conditions the floor was designed to take the stresses caused by a loading of twice the normal, the resultant of which cuts the base at a distance of 2·48 ft. from the toe and causes the upward reaction as shown in the diagram. Under *Q* the resultant cuts the base 3·11 ft. from the toe.

The floor was designed so as to take the varying loads, the first 7·5 ft. from the heel being designed to carry the weight of the fill under a unit stress in steel of 12,000 lb. The next 4·5 ft., which is not subjected to downward pressure, except under the excessive loads, was designed for a unit stress in steel of 16,000 lb. The front portion of the floor was designed to take the upward resultant caused by the loading 2*P*, or twice the normal, the triangle *alh*, Diagram No. 2, *Fig.* 4, showing the loading. The allowable stress under this loading was 24,000 lb.

All unit stresses in the floor, however, caused by the normal loading do not exceed 12,000 lb. In this manner the floor is proportioned so as to develop the strength of the floor and ribs. Under loading approaching the ultimate the upward resultant, of course, becomes very great, and for this reason the change in section between the front wall and floor is made gradual.

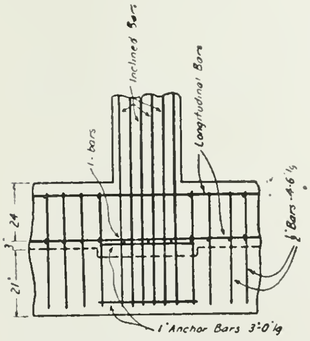


Fig. 5. Details of bars in base.
Dilworth Street Retaining Wall.

Design of Heel Projection.—Under the normal thrust *P* of 17,100 lb., the weight of the wall being 71,000 lb., the necessary co-efficient to prevent sliding would be 0·24. It was assumed that the tangent of the angle of friction would be not less than 0·33. Under the ultimate thrust to be carried by the wall of 47,000 lb., deducting the resistance due to friction of 23,000 lb., leaves a balance to be taken by the heel projection of 24,000 lb. Under the loading of twice the normal, giving a thrust of 34,000 lb., the thrust to be taken by the heel projection would be 11,000 lb. per linear foot; the projection was therefore designed as a cantilever to resist this thrust of 11,000 lb. per linear foot, with unit stress in steel of 16,000 lb. Under the ultimate loading the stress in steel would be 35,000 lb. The reason for carrying down this toe projection was that the piers in front of the wall could not be relied upon to prevent sliding without carrying them to considerable depth, while the back of the wall, being carried into shale excavation, afforded an advantageous and economical method of preventing slipping. The design of the heel projection is shown in *Fig.* 5.

Tie Rods in Ribs.—The vertical tie rods tying the ribs to the floor of the wall were proportioned to carry the resultant downward pressure and transmit the same to the ribs that were carried up through the rib and hooked over the horizontal tie rods. From the theoretical standpoint these tie rods are not necessary, but it was thought that a much more rigid and permanent construction could be obtained. The unit stress in these rods was fixed at 20,000 lb. per sq. in.

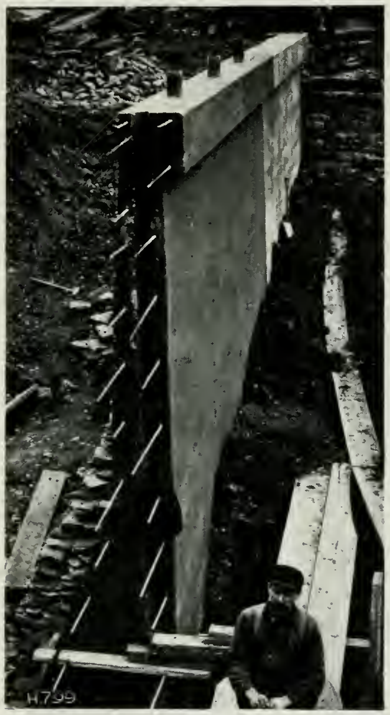


Fig. 6. View in course of construction.
Retaining Wall, Stanton Avenue, Pittsburgh.

The horizontal tie rods were designed to transmit the lateral thrusts from the front wall to the ribs. The unit stress in these rods was fixed at 20,000 lb. per sq. in., the stress being taken at this value on account of the liability of excessive loads over any portion of the wall. At overturning, considering the thrust causing overturning as applied either at $\frac{1}{3}$ or $\frac{2}{3}$ the height, the stresses in these tie rods is about 40,000 lb., but necessarily varies in different portions of the rib. The horizontal tie rods also insure the distribution of the stresses and make secure the bond between the front wall and the ribs. The horizontal tie rods were made in *U* shape, being passed around the structural angles in the front of the wall and hooked around the rib reinforcement in the back of the wall. It will thus be impossible for the ribs of the front wall to separate without tearing out the longitudinal reinforcement in the front wall.

Concrete was mixed in the following proportions :

Plain concrete in piers 1 : 3 : 7 ; reinforced concrete, 1 : $2\frac{1}{2}$: 5.

The gravel used was Allegheny River gravel, conforming to the requirement that it should be graded from fine to coarse, ranging in size from $1\frac{1}{2}$ in. to $\frac{1}{4}$ in.

STANTON AVENUE RETAINING WALL.

This wall was constructed during 1908 and 1909 to replace portions of an old wall which had fallen by overturning. Repairs had been made to the old wall at various times, and several very heavy buttresses or counterforts had been built. The slope in front of the wall is steep, extending 500 or 600 ft. to the valley below. The material is mostly red clay underlaid with shale. The slope in front of the wall had slipped, the surface of the ground showing cracks several feet in depth, these cracks extending down the slope several hundred feet. The old wall seems to have failed on account of the slipping of the material in front of it. The filling behind the wall was found to be a mixture of yellow and red clay, which during dry weather cracks up and freely admits water. There was no provision for drainage in the old walls, which probably contributed largely to their failure.

It was proposed to replace the portion of the wall that had fallen, and with this in view designs were made for reinforced concrete retaining walls of two depths, 20 ft. and 15 ft. respectively. It was proposed, if necessary, to go below this depth, to utilise the stone recovered from the old wall, using such stone as was satisfactory.

After the contract was let it was found impracticable to build the wall without carrying the base to shale. Owing to the fact that the slope in front of the wall continued to slide owing to its angle of inclination, and also to the fact that it was necessary to go a depth of from 8 to 10 ft. under a portion of the 20-ft. section, it was decided to change the detail of the reinforced wall in some respects. The base was increased from 9 to 10 ft., and the dimensions of the wall at other sections was also increased. No change was made in the quantity of steel. The reason for increasing the base to a greater width was to transmit the resultant pressure to the rubble base in a safe manner, and also, to a limited extent, to provide for a depth of pressure exceeding 20 ft.

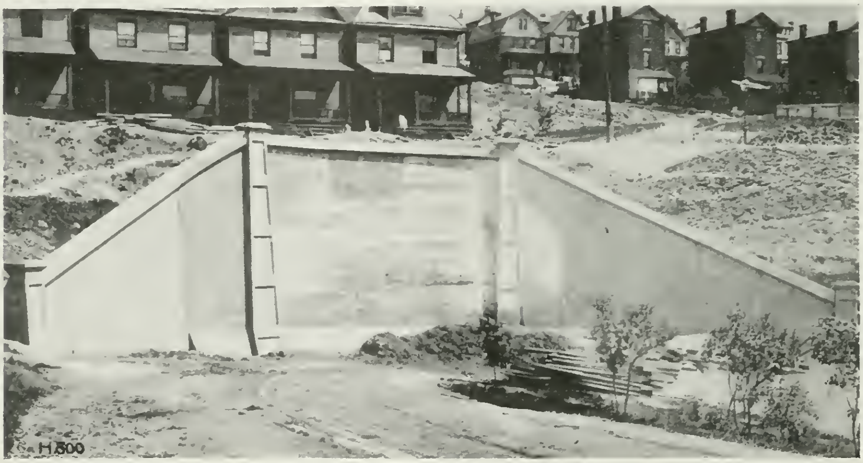


Fig. 7. Retaining Wall, Michigan Street, Pittsburgh.

MICHIGAN STREET RETAINING WALL.

This wall is a heavy reinforced concrete wall of the cantilever type, designed for the surcharge of a new fill placed on a steep slope. The wall was built in connection with the grading, paving, and curbing of Michigan Street, and was built to protect a roadway in McKinley Park. The wall consists of an arched central portion of reinforced concrete with plain concrete abutments or wing walls at the end. From the back of the wall the embankment slopes up to the street line, the angle of the slope varying with the distance of the wall from the street line.

On account of the location of the wall an attempt at ornamentation was made.

The wall was designed for a normal pressure of twice that developed under the assumption that ϕ would equal 37 degrees 30 min, which approximately equals the angle of inclination of the original surface of the ground. The depth of surcharge was about

$5\frac{1}{2}$ ft. The stability and internal resistance of the wall was also investigated under the thrust developed by the forces Q and Q'. The weight of the wall and the superimposed earth was 25,000 lb., its moment of resistance was 231,000 ft. lb. per lin. ft. The steel was computed for stress in steel of 8,000 lb.

In designing the wall the base was spread, the toe being extended 5 ft. on the front of the wall. This extended base was desirable on account of the steep slope of the existing surface and the fact that a mixed clay and earth fill was to be placed.

A comparative design was made for a gravity wall, but on account of the conditions it was thought advisable to build the reinforced wall, since the cost was but very little greater and it possessed a much greater factor of safety.

The vertical rods in the back of the wall, $\frac{7}{8}$-in. rods spaced $4\frac{1}{4}$ in., were extended from the top of the wall to the base. The inclined rods, $\frac{7}{8}$-in. rods spaced $4\frac{1}{2}$ in. centre to centre, were tied to the vertical rods and also to the transverse rods in the heel, which were $\frac{3}{4}$-in. rods spaced 6 in. centre to centre. The ends of the inclined rods were hooked over longitudinal rods in the base. The rods in the extended toe were $\frac{7}{8}$-in. rods spaced 4 in. centre to centre. A toe projection was provided, but was not necessary in construction owing to the fact that the base of the wall was in shale. The shape of the wall also is such that arching action of the reinforced section against the heavy concrete wing walls is provided, and provides a very great factor of safety against overturning.

The proportion of concrete in this wall was 1 : 3 : 6.

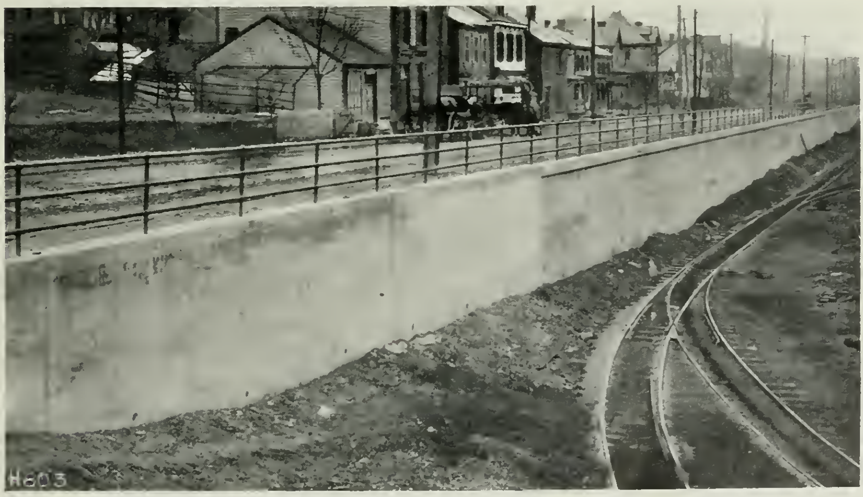


Fig. 8. Retaining Wall, Washington Avenue, Pittsburgh.

WASHINGTON AVENUE RETAINING WALL.

The wall erected on Washington Avenue is a reinforced concrete retaining wall, cantilever type, 500 ft. long, at each end of which a length of gravity wall of less depth was built, the total length of the gravity walls being 375 ft. The construction of this wall was rendered difficult by the fact that a 6-ft. brick sewer had been built near the street line, so that the space available for the base of the wall was restricted.

In preparing the contract plans it was anticipated that the width of base would not exceed 4 ft.

The wall was designed for a base of 4 ft. and a depth of 19 ft. from top of wall to bottom of the reinforced wall. It was anticipated that, where desired, plain concrete would be placed under the base of the reinforced wall.

The concrete was 1 : 3 : 6 mixture, and the wall was built in sections varying from 90 to 60 ft.

The wall was designed for a depth of fill of from 7 to 9 ft., and the angle of repose assumed for the fill was 45 degrees. It was assumed that the excavation through the old fill would exert little if any pressure on the wall. The normal thrust developed, with $\phi = 45$ degrees and $\gamma = 100$ lb. per cu. ft., $= 700$ lb. per lin. ft. of wall. The weight of the wall and the superimposed earth was 9,400 lb. per lin. ft. The overturning moment calculated to the base of the reinforced section for $h = 19$ ft., is $M_t = 9{,}100$ ft. lb. per lin. ft. The moment of resistance of the wall about the toe is $M_r = 18{,}200$ lb. per lin. ft. Under this thrust the resultant cuts the base about 1 ft. from the toe, the base bearing under the thrust being 3 ft.

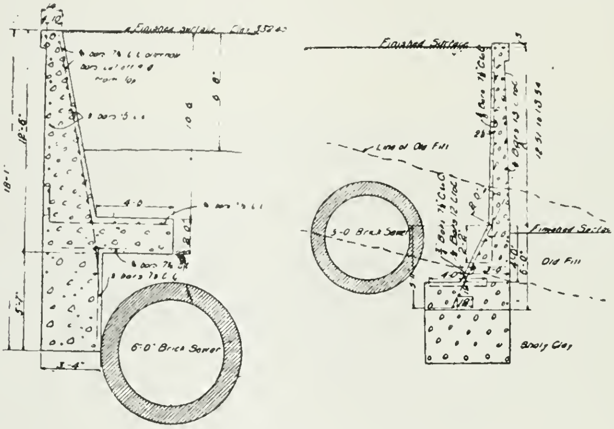


Fig. 9. Sections.

Retaining Wall, Washington Avenue, Pittsburgh.

While this was considered safe for the character of the fill to be placed, which was to be a mixture of shale and clay, it was thought desirable to carry the base deep, so as to provide an additional factor of safety for overturning. For this reason the depth of the wall was carried 6 ft. below the top of the rail. The steel reinforcement was computed so that the stress on the steel, under the normal thrust developed, by the earth, with $\phi = 33$ degrees 42 min, would not exceed 8,000 lb. per sq. in.

When the trench was opened up it was found that the line of the sewer was irregular, and that for a considerable portion of the length of the wall a width of base of from 5 to 6 ft. was available. The base of the wall was therefore made the full width of the trench, and horizontal steel reinforcement was placed in this extended heel, the original design only contemplating a 6-in. offset 2 ft. from the base.

The distance from the bottom of the reinforced concrete section to the surface of the shale varied from nothing to 4 ft. With the exception of the first section of 90 ft. the wall was carried to shale, plain concrete being placed under the base of the reinforced section. In order to utilise this depth of reinforced concrete vertical reinforcement was placed in the back, being hooked over the horizontal rods of the heel.

With a base 5 to 6 ft. in width the wall has a much greater moment of resistance, the moment of resistance of the wall being increased from about 18,000 to about 41,000 ft. lb. per lin. ft.

Section.—The section of the wall consists of a thin section 14 in. wide at the top and 18 in. at the bottom, and from 12·5 to 13·5 ft. in depth. This portion is reinforced with ⅝-in. rods, spaced 7½ in. centre to centre. The next section is 4 ft. in depth, and its thickness is 18 in. at the top and 3½ ft. at the bottom, and is reinforced with ¾-in. rods spaced 7½ in. centre to centre. The transverse rods in the base, which was 2 ft. in thickness, are ¾-in. rods spaced 7½ in. centre to centre. The rods in the upper portion of the wall were cut off alternately 4 ft. from the top.

Expansion and Contraction.—In the first section of the wall, 90 ft. in length, the longitudinal reinforcement, placed so as to provide for temperature stresses and to aid in distributing the stresses, consisted of nine ½-in. rods spaced about 19 in. apart, the percentages of steel in the upper portion being 0·092 per cent. The base of the first section was on hard clay. No cracking of the wall resulted. The next section, however, was carried to rock, and this 90-ft. section cracked into three parts of approximately 30 ft. in length. The percentage of reinforcement was increased, sixteen ½-in. rods being placed in the upper 13 ft. of the wall, the percentage of the steel then being 0·165 per cent. This quantity of the steel seemed to be effective in preventing cracks.

BUILDING TRADES EXHIBITIONS IN MANCHESTER.

Two Building Trades Exhibitions were held at Manchester during the past month—one organized by Messrs. Montgomery, who are so well known in connection with the exhibition held biennially in London; and the other was the Seventh Manchester Building Trades Exhibition, and was held at the City Exhibition Hall, Deansgate. Both were very successful.—ED.

THE BUILDING TRADES EXHIBITION, RUSHOLME, MANCHESTER.

THE Building Trades Exhibition, which was held at Rusholme from March 9th to 23rd, under the organisation of Mr. H. Greville Montgomery, was practically the same, on a smaller scale, as the very successful exhibitions held biennially in London under the same able management.

Floors and Roofing, etc.—Among the exhibitors were the *Kleine Patent Fire Resisting Flooring Syndicate,* who showed floors, staircases, and roofs constructed of reinforced hollow bricks. The framework of their stand consisted of steel stancheons and horizontal steel joists, covering a round area of 20 ft. by 10 ft. The floor of their exhibit was raised about 3 ft. from the floor level of the hall, in order to show two short flights of stairs being constructed on the Kleine system to lead up to same. This floor was constructed of small size hollow bricks, set on edge and reinforced with flat tension bands to form a floor 6 in. deep, spanning 10 ft., and built into the webs of girders on either side only, the ends not requiring a support. Above this was constructed a 20 ft. span 7¼ in. thick, built with Kleine patent hollow bricks, with closed ends, reinforced with round rods and cantilevering slightly beyond the steel frame. A small opening was left in the middle of this span without trimming hoists, to demonstrate the fact that openings may be left if desired. On top of this 20 ft. span were built Kleine Hollow Brick reinforced walls. The 20 ft span was built on top of the girders to demonstrate that there is no arch action in the Kleine floors, and consequently there is no thrust on the steelwork.

Messrs. G. M. Callender & Co., Ltd., included among their exhibits their "Callendrite" reservoir sheeting, which is largely used for making reservoirs, filter-beds, swimming baths, waterproof tanks, etc. When this sheeting is used the inner slopes of the embankments and the floor of the reservoir are lined with a thin layer of concrete, so as to form an even surface for the sheeting, and then the latter is covered with 3 in. of concrete as a protection from mechanical damage. A model was shown of a reservoir lined with Callender's sheeting to demonstrate its impermeability. Other specialities shown by this firm were "Ledkore" dampcourse for foundation work, "Permalithic," asbestic, jointless flooring, which is suitable for warehouses, hospitals, etc. It is claimed for "Protex," damp-proofing emulsion, which was also shown at this stand, that it renders, when applied cold with an ordinary brush, porous concrete blocks damp-proof.

Reinforced Concrete.—*The "Moseley" Construction Co.'s* stand attracted a good deal of attention, as it is the first time this company's specialities have been shown to the public. The front exhibit on the stand was a Terracrete wall and doorway, a combination of terra-cotta and reinforced concrete. The former is made in hollow troughs with large perforations in the bottoms, which are built course by course like masonry and filled with concrete. Reinforcing rods are laid in horizontally and vertically, as usually done in reinforced concrete work, and a monolithic structure is secured. No shuttering is required, and the terra-cotta front and reinforced concrete walls go up simultaneously. Another wall was shown where the terra-cotta, blue vitreous tile, white glazed and coloured faience work were hung on and built in with reinforced concrete troughs. Other exhibits of this firm consisted of a reinforced concrete wall built with a cavity—the concrete troughs having a division to effect that purpose. A beam constructed of the hollow troughs was shown spanning the length of the stand with tension rods inserted. These beams can be lifted up into position and filled with concrete in situ, saving all centering and shuttering. Some 4½-in. walls were shown with artificial granite finish and having great strength, all built on the same system. A waterproofed concrete flat roof and a water tank were also amongst the exhibits on the stand, and a pier, showing the method adopted in reinforcing the concrete troughs and connecting the foundations slabs, connections from pier to pier, counterforts beams, etc.

The Moseley system is, we understand, to abolish the use of all shuttering and centering and substitute the hollow concrete trough. This is made of a strong aggregate, and in fixing gives great strength to the wall or beam. Rapidity of erection and fine artificial stone finish are other important features of this system.

Piling.—*Messrs. J. and W. Stewart* exhibited their C.P. (cast in place) method of concrete piling, which may be described as follows:—

A heavy steel tube, generally 16 in. outside diameter, is placed on top of a loose cast iron point and driven into the ground (until the required final set is obtained) by means of a heavy drop hammer operated by a high-speed friction winch. The tube is filled with concrete to a height of several feet (according to length of pile) above the level at which it is desired to finish the head of the pile. A powerful pulling tackle is coupled on, and the tube is slowly and steadily drawn out of the ground. As the tube is withdrawn the concrete sinks and expands, filling up tightly the hole so formed.

The Ransome ver-Mehr Machinery Co. also showed various sections of their Ransome Interlocking Piling, including a new corrugated section with rivetless interlock (Type D). This particular section is specially useful for all classes of cofferdam work, and, in addition, a sample was shown of a combination of this section, built up to form bearing piles. A large quantity of these piles have recently been supplied to the Rosyth Naval Base.

Mixers.—The only concrete mixers exhibited were those of the Ransome ver-Mehr Machinery Company, who showed all their well-known types, including their Ransome Mixer No. 0 size, with a batch capacity of 5 cu. ft., complete with automatic water tank and charging measuring hopper, with direct coupled electric motor and mounted on road wheels for convenience in street pavement works, for which this mixer is much used. It is also greatly employed for reinforced concrete work.

Bar Bending Machine.—*Mr. W. Kennedy*, of West Drayton, gave demonstrations of his portable bar-bender for reinforced concrete. This is a hand-power machine which bends acute and obtuse angles in mild steel rods up to 1½-in. diameter cold.

Tiles, Slabs, etc.—Among *Messrs. Bells' United Asbestos Co.'s* specialities were their "Poilite" asbestos cement roofing tiles and building sheets, and their exhibit was a practical demonstration of the adaptability of that material for engineering works and every kind of building where a light, fireproof, and permanent roofing material is required; and that it is not lacking in artistic qualities was plainly demonstrated. The firm also showed their different Asbestos Fireproof Paints for the protection of woodwork from the inception and spread of fire; also their Ferric Oxide Paints for bridges, girders, and ironwork generally.

Messrs. G. R. Speaker and Co.'s pavilion, with "Eternit" slates and sheets, was a wood-framed building which was otherwise wholly constructed of "Eternit," showing how this material can be used for roofs, walls, ceilings, and imitation timber work.

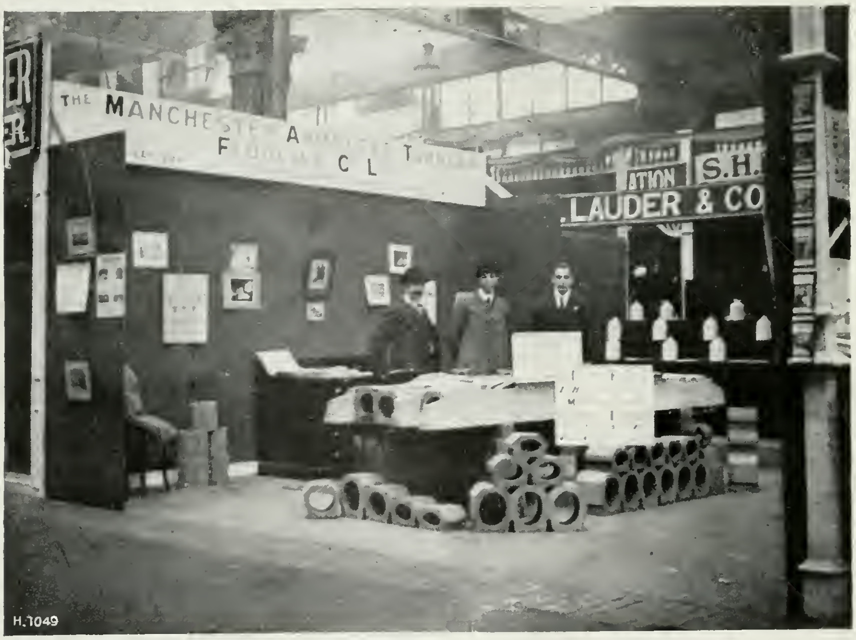
Messrs. Machin and Koenig had a stand showing how their H.P. asbestos tiles and slabs and other specialities can be applied for the construction and decoration of ceilings, interior and exterior walls, panelling, etc.

At *Messrs. Frederick Jones and Co., Ltd.'s* stand their "Hercules" Fireproof Partition Blocks were shown in various stages of fixing and finishing. These are supplied in Portland cement concrete, plaster concrete, or pumice concrete. The exhibit also showed their Plaster Ceiling Slabs.

BUILDING TRADES EXHIBITION, DEANSGATE, MANCHESTER.

Another Building Trades Exhibition was held in Manchester from the 5th to the 16th March at the City Exhibition Hall, Deansgate.

The only system of reinforced concrete exhibited there was that of the *Manchester Armoured Tubular Flooring Co., Ltd.* Samples of the tubes for making this flooring were on view at the stand, as will be seen from the accompanying illustration.



Manchester Armoured Tubular Co.'s Stand.

These floors are formed of concrete webs or joists containing steel reinforcement with concrete tubes between, the upper surface of the latter forming the top layer of concrete and the lower surface the soffit for plastering. Armoured tubular floors can be constructed for spans up to 30 ft.

The Christie Patent Stone Co., Ltd., were exhibiting samples of their steps in various qualities and textures, also balustrades and garden ornaments, sills, mouldings, etc.

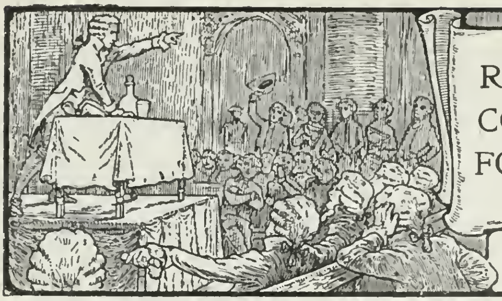
The T. L. Smith Co. were showing a "Smith" Mixer with feeding hopper driven by an electric motor, also a "Smith" hand-driven concrete mixer and a "Victoria" concrete mixer mounted on truck and fitted with side loader driven by an electric motor.

Among the other exhibits were to be seen "Rok" Roofing, made by *Messrs. D. Anderson and Son, Ltd.* Also their "Storiflex" felt and "Bitumen" sheeting; *Messrs. F. McNeill's* asphaltic roofing felt for exterior roofing, and a thinner asphalted felt for temporary structures.

Bells' United Asbestos Co. also had a stand at this exhibition, showing their "Poilite" roofing tiles and "Poilite" asbestos sheets.

Vulcanite, Ltd., had on view at their stand a model brick building with a flat roof tastefully laid out as a miniature roof garden, the roof being constructed of joists and 1-in. T and G boards covered with patent vulcanite roofing. They also showed a water tank formed of 9-in. brick walls, the bottom and sides being lined with vulcanite and afterwards bricked in with white, glazed bricks.

Messrs. Roneo, Ltd., were showing their steel doors, which are suitable for private and public buildings where fire-resisting material is used. They are panelled in any style desired and finished in art colours or grained. Steel partitions and library shelving, etc., were also to be seen at this company's stand.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

THE CONCRETE INSTITUTE.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE CONCRETE INSTITUTE.

HIGH DAMS OF GREAT LENGTH.

THE MASS GRAVITY TYPE & THREE TYPES OF SPAN DAMS.

Paper by REGINALD RYVES, Assoc.M.Inst.C.E.

The following is an abstract of a paper read by Mr. Reginald Ryves before the Concrete Institute on March 14th. An interesting discussion followed, a summary of which is also given below.

This paper was based on a study of high masonry dams of such length that they might be regarded as dams proper, and in no sense as structures partly held up to their work by the sides of the river valleys.

The paper was confined to those connected with the building of dams of a great length, the whole of which length was more than 140 ft. clear of the ground, and a part of which might be as much as 180 ft. or more to hard rock. The height of 185 ft. clear was adopted for purposes of calculation. The subject of the paper was, further, kept within somewhat close limits by the condition that the highest calculated maximum stress in the masonry must not exceed 12 tons per sq. ft., at any rate, by Bouvier's computation, though by Unwin's computation a somewhat more severe stress might be allowed.

There were four distinct types of dams for wide valleys, and entirely excluding the dam which was a single arch, in plan, and the dam which was, in its lower part, practically a plug in a gorge. The four types were:—

1. The mass dam proper, designed on the ordinary theory.
2. The parallel slice dam, designed in exactly the same way, but with more concentrated water loads.
3. Captain Garrett's type, or the dam of arches in which the weight of the arches was partly taken into account, but the dam otherwise designed as a gravity dam with concentrated water loads.
4. The author's proposed thrust buttress dam of arches, in which the whole of the water load was taken by masonry in direct compression, and neither the weight of the buttress nor the weight of the arch was taken into account as regards stability, except for resistance to sliding bodily when the ground was comparatively soft.

As regards the first class, the mass gravity dam, it was found that for high dams, above 150 ft., the maximum stress increases very rapidly indeed, and that to build a dam 200 ft. high we must exceed a stress of 12 tons per sq. ft. by Bouvier's computation, and greatly exceed it by Unwin's computation.

In the second class, which was one type of the dam of spans, it was found that its height was limited by two considerations. First, one could not build truly parallel slices of more than some limiting height without side buttresses. If the sides were sloped the strict theory no longer held good, and we were logically led to Captain Garrett's type, in which the weight of the arches was partly counted upon for stability.

The other limiting factor was that the concentration of water loads in these separated slices increases both a and ϕ so much that we got very high maximum stresses.

Since the weight of the water apron was not taken into account at all in this type, it was one which was adapted for use with thin arches of very strong stone, or a reinforced concrete beam and slab arrangement, for the water apron.

The third type, in which the weight of the arches was partly taken into account, had been successfully used for the Agar dam, in Rajputana, a dam 68 ft. high, designed and built by Captain A. ff. Garrett, R.E.

The indeterminate nature of the stresses was, however, against this type for great heights, but with small gaps and large buttresses it had certain advantages. For instance, with the bases of the buttresses merging together it would be a very good type for through-sluices, or for a suitably arranged overflow.

We now come to the fourth class, the author's typical design for a thrust buttress dam. The best slope for the water face was, under normal conditions, 45°, and the dam then consisted of inclined arches, of increasing thickness with increasing depth sloping at 45°, the abutments resting against the up-stream face of the buttresses, which were built up of layers all inclined at 45°, the thickness depending upon the material used, and the width of each layer being such that the thrust upon it was the maximum allowed thrust.

The calculation of thrust was made by adding the water load transmitted to any plane parallel to the face to the resolved part of the weight of masonry above the plane in question, the same method of calculation being adopted in every part of the buttress. The resolved part of the weight of the arch was in each case added to the water load. The water loads were thus transmitted by direct thrust, the layers not being bonded together, but left free to take up the deformation under the water load by sliding on one another. If the rock would not take the load at an angle of 45°, the water face might be made flatter and longer, and the buttresses, always at right angles to it, steeper and shorter. The typical design was, however, that with a 45° slope. Every part of the dam was subject to the same stress except that the top layer of the buttress and the upper part of the arch ring might have respectively a greater width and a greater thickness, the minimum in each case for the materials used.

This adoption of a general stress, instead of one varying from nearly zero to some maximum, was an important element of economy.

Such a dam might be built of materials which were not suitable for the rubble masonry or mass concrete of a mass gravity dam. For instance, slabs of limestone or hard shale may be used, both in the arches and in the buttresses, with their bedding planes at right angles to the direction of thrust, or it may be built with blocks of concrete. On the other hand, it could be built of rubble masonry, or the layers could be built up from their lower ends, of rammed concrete between rubble walls.

The definite propositions which the author puts forward are:—

1. That there is no precedent for the building of a mass gravity dam in an unassisted length more than 150 ft. clear of the ground.

2. That our present ideas as to maximum stresses allow of the building of such a dam for heights only slightly in excess of this.

3. That since the improbability of the stress actually reaching $p \sec^2$ increases as the slope at the down-stream toe is flattened, we may, for heights above 150 ft. and for angles flatter than 45°, allow one maximum computed stress for $p \sec^2 a$, and a computed stress for $p \sec^2 \phi$, some 50 per cent. higher. This applies both to mass gravity dams and to parallel slice gravity dams, and perhaps also to Captain Garrett's battered buttress type.

4. That interrupted dams, the gaps being spanned by horizontal or inclined arches, or by beams, offer the important advantages—

(*a*) Economy of material.
(*b*) Absence of contraction cracks and temperature stresses.
(*c*) Maintenance of waterway during construction.
(*d*) Cheapness of sluices.
(*e*) In some cases a wider choice of materials and of methods of building.

(*f*) And that the interrupted dam, or dam of spans, may be considered as an alternative to the mass gravity dam for great heights, especially as regards the height clear of the ground.

5. That the author's design for a thrust buttress dam is suitable for heights up to 200 ft., with materials allowing of a compression stress of 10 tons per sq. ft. and up to 300 ft., if a stress of about 16 tons per sq. ft. be allowable.

6. That none of the dams at present built are thrust buttress dams, either as regards the method of building or in practical effect.

7. That in order to avoid tension near the up-stream toe of a high mass dam, or of a gravity buttress in a dam of spans, it is desirable to make the connection with the rock somewhat as follows:—

The up-stream third of the base a flat surface with no bonding and *no trench*.

The middle third bonded, but not deeply trenched.

The down-stream third effectively bonded with the rock cut in faces at right angles to the thrusts of the masonry.

8. That when a high dam is to be built with a large part of its total height from foundations below the bed of the reservoir, it may be worth while to relieve that part of the upstream face from water load by directly draining it through the dam.

9. That dams in the tropics, or near the tropics, with down-stream faces towards the Equator, or facing east or west, may with advantage be provided with temperature skins to a depth depending on the extent to which daily or annual changes in temperature deprive the outer shell of the face of its value as material resisting compressive stress.

10. That a long and high mass dam may usually be, with advantage, built with gaps in the masonry across the length, these gaps being filled when the temperature of the dam is judged to be nearly at or a little below its future mean temperature. If cracks are specially to be guarded against, the gaps may be filled at the end of the cool season and after the temperature of setting has subsided—that is, when the average temperature of the masonry is at its lowest.

11. That the grading of material in a large dam should be such that the large stones are not " plums " or displacers, but play the part of the stone in ordinary concrete.

DISCUSSION.

Mr. Charles F. Marsh, M.Inst.C.E. —He should like to ask the author if the parallel sliced dam was formed of buttresses with parallel sides, in contradistinction to those with wedged-shape buttresses, and that therefore the arch was always of the same span all the way down. He did not quite understand how the arches of the parallel sliced dam abut on to the slices.

The form of dam which the author advocates was new to him, and it appeared to be an extremely good form of dam and an extremely economical one.

The author stated that a reinforced concrete dam could not be built to any considerable height; but he would like the author to state the limits to which the ordinary type of reinforced concrete dam with a slab and buttresses could be built. The great advantage of a reinforced concrete dam, of course, was this: that there was absolutely no pressure on the other side, because, of course, you could drain if there was a wall along the toe down to the impervious strata; any upward pressure was relieved at once by letting it drain through the base of the dam and so down stream. The other advantage was that the pressure on the foundations was evenly distributed throughout. It was not in the form of a triangle; it was almost in the form of a rectangle. In the case of the dam described a similar thing occurred at the buttresses. He wondered if the author had taken into account the vertical component of the pressures on this dam. With the height of the dam, of course, there would be a certain weight of masonry underneath the apex which would be distributed over the whole width of the buttresses, and there would also be a considerable pressure due to the water on an inclined face of 1 to 1. Although the pressure was evenly distributed by having a flatter slope on the water face it was also greatly increased; the vertical component at any rate was very greatly increased. He did not know whether he had ever taken out the vertical pressure which he would get vertically under the apex of the dam. There were one or two more things he would rather like to know. One was as to the draining of the upstream face below the bed of the river or the surface of the ground. He did not know what the conditions were in India, and there might be a waterproof sealing of the surface of the ground very soon; but in England it would be an extremely dangerous thing to put drains through at the bottom of your dam—it would probably empty your reservoir. He supposed that would not occur in India.

He also did not quite understand what was meant about plums. He had always thought that the great thing to be aimed at when putting in big blocks of stone in concrete as plums was to keep them apart and properly surround them with the ordinary concrete, the concrete mixed in the proper proportions to prevent leakage of water. It was an extremely economical thing to do to put in large blocks of stone, and the concrete was none the worse for it; but he should have thought that those stones should certainly be kept from four to six inches apart at the least.

Mr. Ewart S. Andrews, B.Sc. (Lond.).—His knowledge on the subject of dams was restricted to the theoretical consideration of the case. He had had no experience of the practical construction of them, but had the good fortune of being at University College some eight years ago, working with Professor Pearson at the time that he was working with Mr. Atcherley on the subject of the stability of dams; and, as they were probably aware, the result of the investigation was a paper which raised a very large amount of discussion amongst those interested in the construction of dams. The whole of the ordinary theory of dams was based upon an assumption that the masonry was an elastic material, which assumption was probably considerably far from the truth. As far as he knew, nearly all the dams that had been designed had been based upon the well-known law of the middle third, and it had always seemed to him to be a very happy accident if it happened to agree with the result in practice. It was on account of the extreme difficulty of the satisfactory theoretical treatment of the ordinary gravity dam that they should be rather careful how they approached the subject.

He was rather surprised that no reference had been made to the consideration of the stresses in dams, based upon the points which were raised by Professor Pearson in the paper to which he had referred. That paper was principally to direct attention to the tensile stress in the toe of the dam, caused by the upward pressures acting upon the toe as a cantilever and thus causing tensile stresses in vertical sections. Later on, Professor Pearson carried out some further experiments on jelly models of dams; and, although they found very great difficulty in getting results that were sufficiently consistent to enable them to initiate any scientific theory, their result was that one of the most important points to allow for was the bonding of the dam to its substratum. He would like the author to say whether he considered Professor Pearson's investigations insufficient in that respect.

Mr. E. Fiander Etchells, F.Phys.Soc. — In regard to Garrett's design, there was one point in Garrett's design which he would like to draw their attention to, because it was not always attended to in the arched dams. Garrett had his arches at the arc of a circle. That, he thought, would be quite correct, because the pressure of the water was radial in every direction, and if you could complete the arch you would get a circle of radial pressures in which the lines of stress would be running in a curved form. Sometimes the design would be rather too flat, as though the pressures coming upon them were parallel to one another instead of being radial, and the effect of the radial pressures was manifested to develop in the lines of resulting pressure.

Mr. S. Bylander. — One point that very often was forgotten in considering the right theory to apply in connection with the dam was this, that many assume a dam to stand on rock which is not elastic and which will not give under pressure to any considerable extent. This was, of course, not the case. The rock would give or change its thickness in a similar way as a structure subjected to stress would. It could be demonstrated very nicely and very simply by making a section of a dam in jelly and subjecting one side to pressure. Then it could easily be seen how the jelly was deformed and showed the intensity of the pressure. Suppose you took a section and drew a horizontal and vertical line, thus producing squares, then by producing a pressure on one side of the dam the squares would be deformed, and the more they were deformed the greater the pressure. There must be a difference in pressure at different layers.

Monsieur Maurice Béhar. — He did not see any reason why reinforced concrete was not suitable for high dams. He thought high dams in reinforced concrete could be done all right. They certainly would be very much cheaper. The second question would be the question of excavations. By the distance between two buttresses the excavations would be reduced considerably. It might be that in high dam construction, if reinforced concrete was adopted, the cost might come down perhaps 50 per cent. He personally knew works of that kind. There was one called Barrage de Furens, near St. Etienne, built by M. Hirsch. Professor at the Ponts et Chaussés School. While he was receiving his education in engineering there M. Hirsch built a dam for the distribution of the waters for a canal going from the Rhone. This dam was done in masonry, and this masonry was working in many parts in tension by a kind of arrangement. He did not remember exactly which way, but if you compared the size and the thickness of the wall at the bottom with any dam erected in other parts of the world the thing was absolutely an exceptional one. He thought the figures were 7 or 8 metres at the bottom for a height of 50 or 60 metres of water. The dam had existed more than thirty-five years.

Mr. A. Alban H. Scott, M.S.A., M.C.I. — With regard to reinforced concrete dams, the author simply made the statement that they cannot be done to a great height. He did not give the reasons, and he (Mr. Scott) thought, with Monsieur Béhar, that there could not be any reason if you can build in slab concrete to any height why you cannot build in reinforced concrete, except as relaying reinforced concrete upon steel for certain stresses for getting such enormous heights, especially in a place like India, the question of variations of temperature, the expansion of the dam might entirely throw out all one's calculations for portions of it. But that, after all, was perhaps only a question of the thickness of the protection to the steel. But from an engineering calculating point of view he did not see why there should be any difficulty.

In a very small little dam, the first one he ever erected, they put up one of 15 ft. high, exactly on the same principle as that. It had answered quite well; they get sudden shocks of water on it. He did not see why that should not be continued up another ten or fifteen or twenty times that height.

Mr. W. Perkins.—The author had stated that it was impossible to construct a dam in that form with reinforced concrete, and other speakers had dealt with that point. He wanted to ask if it would be possible to construct a dam of that section in a compound way by using masonry for one portion of it—the buttresses—and by using reinforced concrete as a slab instead of the arches?

Mr. R. Dudley, Assoc.M.Inst.C.E.—The author had treated the substratum on which the dam rested as if it were known before he had designed the dam. In a dam that he was connected with in South Wales they made some beautiful calculations, but when they came to excavate the rock and drill holes to strike a vein of sand going down about 50 ft. in one place, and a little bit further on they struck on an old coal lift, so that any calculations made were based on the assumption that you were going to meet rock and strata. He thought Mr. Ryves was going to treat the middle third in one way and the up-stream third in another and the down-stream third in another, tension and bonding, and not bonding, gradually increasing bonding from the front to the back. Well, if a vein of top sand was struck, and you had to go down to your concrete or your puddled trench, it would be very interesting if the author would kindly explain how he would get over such a state of things as that.

The Chairman (Mr. E. P. Wells). — He personally did not see why reinforced concrete should not answer with the heights shown, and also give a very considerable saving in cost. The suggestion that he would like to make himself was this: that for a height of 100 ft. up, instead of the author's thrust buttress being at a centre of 60 ft., he should bring it to 30 ft. centre, so that the arches on the face, to resist the water pressure, would be a 30 ft. central span, as against 60 ft., and so only have a quarter of that thrust. It then became a question as to whether the extra amount of material that would have to be put into the buttress would cost more than the extra material put into the arch for a 60 ft. span as against a 30 ft. span.

One practical suggestion he would like to make was that if the author constructed a dam of that size again, instead of having a perfectly plain surface on the face, that it should all be in a series of steps of about 18 in., because, in erecting, you could carry the whole of the scaffolding along with perfectly level bottoms, and it could be lifted up from the top to the bottom, and the same scaffolding used throughout the entire length; whereas, if you had a plain surface that anything could slide upon, it became a most difficult matter for construction and very much more costly. Instead of having the face of the dam at this angle of 45 degrees, step it all the way down, and it follows that on the same scaffolding, with all your cranes on it, you could do the whole of the work from one end to the other. As the other series is constructed, so it could be lifted up.

With regard to the question of reinforced concrete, he did not see any reason why it should not be done in that material. In fact, the stresses that the author showed on the stonework did not amount to more than, he thought, 10 tons a foot on the base. It was a very poor plain concrete that would not resist the ten tons a foot, and even the London County Council permit twelve, so that if we put this in reinforced concrete the cost could very considerably be reduced. In fact, if the author were to go into the matter carefully, even with the increased cost of cement in India, he would find the maximum total weight that he would have in his foundations, if in reinforced concrete work, would possibly not exceed five to six tons per square foot, especially when you could take into account that, if he worked his buttresses at centres of about 60 ft. and inverted the arches in between the sixty, he had got an enormous area over which to distribute his load.

The Author's Reply.—He might say first that he had not made calculations to show that a reinforced concrete dam up to 280 ft. high on the principle of his thrust buttress dam could or could not be built. He said that reinforced concrete could not be used economically for very large dams, and he had no doubt whatever that there was a limit in height under ordinary circumstances beyond which the reinforced concrete dam became too expensive simply because the materials were too expensive.

Under different circumstances, of course, it was possible that a reinforced concrete dam could be built to a great height; and such a case was that of the Furens dam, which was 183 ft. in height and was built to a radius only 828 ft. in a narrow gorge. A dam of that sort might very well be built of reinforced concrete; but his paper was really confined to the consideration of the case of a dam across a wide valley. An enormous quantity of material was wanted and you must spread out your loads upon a large base, and you did not want to concentrate your loads. If you have a material which would carry easily 20 tons to the sq. ft., and by means of that material you transmitted loads down to the ground and then had to spread it all out again to bring it down to the tons per sq. ft., that is an expensive process. It can very readily be done for security and safety of calculation, but it was an expensive process.

It seemed to him that the thrust buttress type of dam hits the happy mean in collecting the water loads and concentrating them only to a small extent, so that it only had to spread them out again to a small extent when it reached the rock, or not perhaps spread them out at all.

Mr. Marsh had asked whether his parallel sliced dam was intended to be on the theoretical side, so that the arches are all the same span the whole way up—the arches or spans, whatever it might be. That was so. The point in connection with that type was that you avoided going away from the ordinary theory altogether. If the theory would hold in the case of a buttress with the sides exposed—vertical sides—you did not want to tempt providence any further by differing the size or by altering the shape in any way; you wanted to treat it as if it were a portion of an ordinary mass gravity end.

Then, as to the reinforced concrete, he had been told that small reinforced concrete dams had been built and not thrust buttress dams because they were calculated as gravity dams. Those who were interested in the matter would find, after going into it, that no dam had been designed as a thrust buttress dam. Several dams in America had been designed with the same relations being placed upon the fact that the part of the water load was transmitted to the rock directly; but those dams were designed as gravity dams on the toppling-over theory. The other portions of them were not so placed that they transmitted water thrust direct to the bottom. Even small dams did not quite act as thrust dams: they transmit the lower part of the water load to the rock direct, but not the other part of it. As to Mr. Marsh's other point, about keeping the plums apart, he certainly did contemplate keeping them quite a foot apart.

In working out the question whether there was tension in the under side of the dam or not, he went on modifying the straight line distribution of pressure until he found there was no tension, but adopting the straight line theory of distribution of pressures under the dam found that instead of working in horizontal layers the usual way in which you worked out the stresses in a mass dam by dividing it into imaginary horizontal layers and dividing it into imaginary vertical layers, he found definitely that he got strong tension, and modified his curve of pressures until he got no tension.

As regards the bonding into the rock on the upstream side, he did not believe in making a trench on the upstream toe at all, because that caused tension. He worked out a number of mathematical examples in the same way, working imaginary theoretical strips, and assuming the dam was fixed at the upstream toe and afterwards assuming it was fixed at the upper half, middle half, and towards the downstream end. If you worked it out graphically, you get very severe tension in the under side of the dam, which was only natural.

As for building high dams in reinforced concrete, he had not worked out examples, but his point was that those who think they could be built had better work out the examples.

As to what he should do if he came upon a deep crevice in the rock, he would fill the crevice up with masonry and reduce the surface as far as possible to a surface similar to the rock. But if it was near the downstream toe, he would bury it right into the dam. If near the upstream toe, he would fill it up and lift it up to the level of the rock as far as possible. But, of course, with a very irregular bottom, you could not evolve any theoretical ideas to make your foundations in that case. You take every advantage of every case which is more or less at right angles to your thrust and use that.

As to the Chairman's point about the buttresses being close together, the buttress was 38 ft. wide in the front, and that means that the span of the arches was very small.

The Meeting then terminated.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE BRIDGE AT UNIVERSITY COLLEGE, CORK.

A REINFORCED concrete bridge has recently been erected over the River Lee in the grounds of University College, Cork, from the designs of Mr. J. H. de Warenne Waller, B.E., who is now acting as Assistant Engineer in the construction of the reinforced concrete bridge over the Suir at Waterford.

The bridge is 72 ft. long over all, 68 ft. between abutments, and gives a roadway of 16 ft. wide. Its artistic appearance will be seen from the view given in the frontispiece to this issue. It is of the girder type with panelled sides and latticed parapets.

The reinforcement consists of plain round mild steel bars. The main reinforcement in the girders was placed in two layers separated by 1 in. of concrete to make provision for the transference of stress from the steel to the concrete; the bars in the upper layer were bent up into the slab at various points as soon as the bending moment diagram permitted. Special bars were used over the centre support to provide for continuous beam action. The bars in the lower layer were allowed to overlap 6 ft. to guard against tensile stresses that might occur should any settlement take place in the central pier. All the main bars terminated in crooks to provide efficient anchorage.

The concrete was mixed in the proportions of 1 : 2 : 4. The lattice panels of the parapet were cast separately and afterwards fixed in position.

The contractor was Mr. Antony Gaul, of Cork, who carried out the work to the satisfaction of all concerned.

Since the completion of this bridge and the long span (40 ft.) floors, stairs, lecture galleries, etc., in the new Science Laboratories of the College, which were also carried out in reinforced concrete to Mr. Waller's designs, the degree students under Mr. H. C. Johnson, the present demonstrator, have designed and followed the construction of a new floor in reinforced concrete for their drawing office, with stairs in the same material to the floor below. It will therefore be seen that, having once proved the advantages of using reinforced concrete the authorities of University College are continuing to use this material wherever possible.

REINFORCED CONCRETE BRIDGE AT GOODWICK.

THE bridge, an illustration of which is shown on this page, has a skew span of 20 ft.

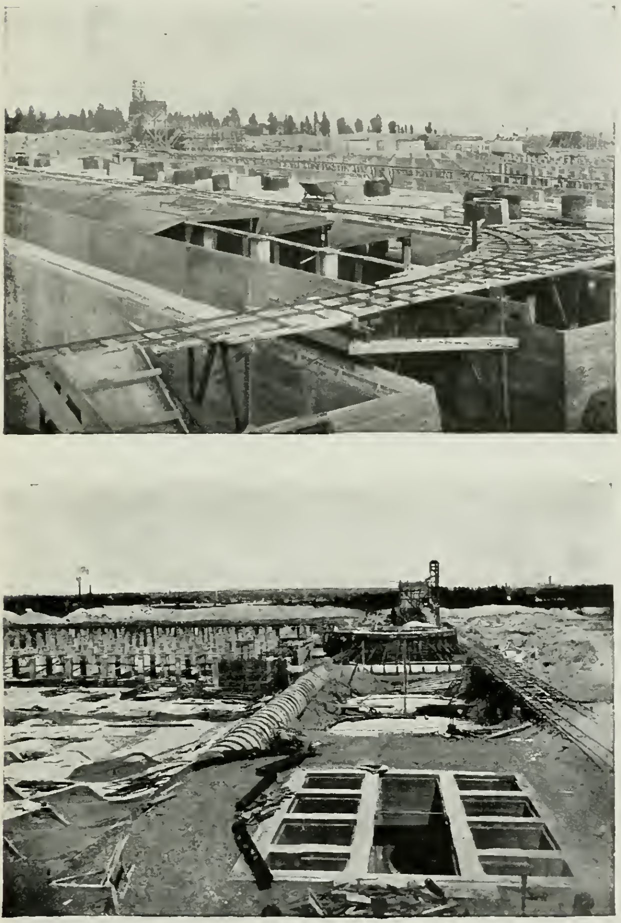
REINFORCED CONCRETE BRIDGE AT GOODWICK.

and is 20 ft. wide between parapets. The deck, parapets, and piers are constructed in reinforced concrete on the Hennebique system.

The deck consists of a 5-in. slab, with seven 8-in. by 14-in. beams under, reinforced with ordinary round steel bars and hoops, in accordance with the Hennebique practice. The abutments were constructed in mass concrete.

The bridge was carried out in two halves, to prevent, as far as possible, any interruption of traffic. It was tested with a 25-ton moving load with very satisfactory results.

The contractors were Messrs. Hobrough & Co., of Gloucester.



Concrete Filtration Plant, Centre Island, Toronto.

A CONCRETE FILTRATION PLANT AT CENTRE ISLAND, TORONTO.

The filtration plant which has just been completed at Centre Island, Toronto, is constructed practically entirely of concrete and covers more than twelve acres.

The filter beds are 312 ft. by 117 ft., six of them being located on either side of a central court, in which is situated the regulating houses, the entrance houses, the mechanical sand washers, and the sand storage bins, together with the great number of pipe lines varying in diameter from 72 in. down to 24 in. At one end of one row of the filters is located a pure water reservoir 312 ft. square, from which the filtered water is taken to the city. All of the filters and the reservoirs have concrete groined arched roofs, inverted groined arch floors, concrete piers, division walls, and outer walls. The piers in the filters are opened 13 ft. on centres and are square. In the pure water reservoir the spacing is the same, and an idea as to the large number of piers erected in this entire work may be obtained by observing that in the reservoir there are 529 and 2,208 in the twelve filters. The height of the reservoir is 13 ft., while the walls are 14 ft. and the average 3 ft. in thickness, the corresponding figures for the filters being 11 ft. and 3 ft. 3 in. For purposes of ventilation concrete manholes, made with the steel forms, have been erected in the centre of every other square of the columns, being spaced 26 ft. centres both ways, and it should be noted here that the facilities for placing material inside of the filters were increased by their use.

The concrete was generally mixed in proportions of one barrel of Portland cement, 7 cu. ft. of sand, and 12 cu. ft. of ballast, ranging from ¼ in. to 2 in. By an endless cable, side dumping cars were hauled from the lake shore, where were located the sand piles and the stone screens, to either one of two Ransome mixing plants. Here the material was elevated by tower arrangement into the bins over the mixer superhopper. Concrete was discharged from the Ransome mixer on to a platform with chutes leading to the cars. With this arrangement continuous mixing was possible.

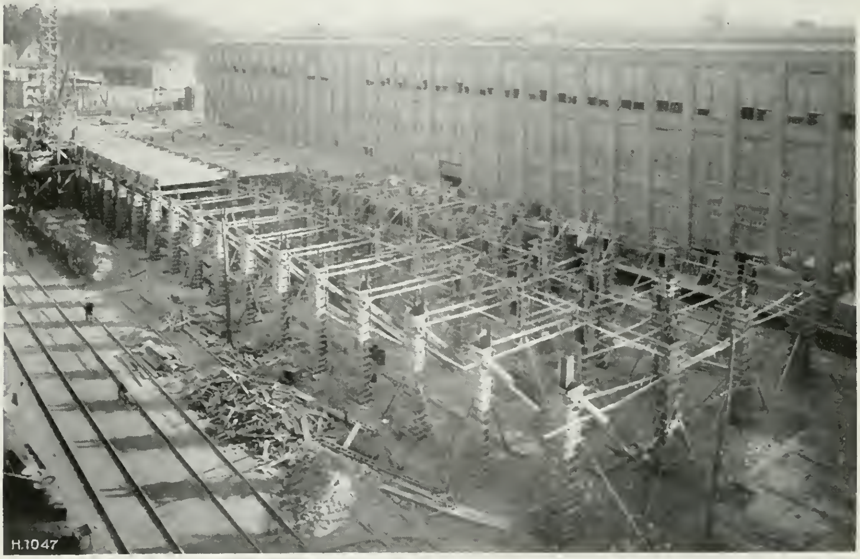
As to the general operation of the filters, it may be said that from the low-lift pumping station located on the lake shore side of the filters the water is forced through a 72-in. reinforced concrete pipe, from which 36-in. reinforced branches are taken off at right angles to the inlet chambers under each entrance building, each branch supplying two filters. Upon entering the filters the water is spread over the surface of the sand to a depth of 36 in. to 48 in., and then successively passes through 3 ft. 6 in. of specially cleaned and graded sand and three layers of broken stone and gravel.

After percolating through these materials the filtered water is collected in 10-in. split tile pipes, laid in parallel rows at intervals of 13 ft., at right angles to, and emptying into, a main drain concrete box 6 ft. wide. These drains are covered with a reinforced concrete slab and empty into 24-in. cast-iron effluent pipes, one of which leads from each filter to a regulator house. Each of these pipes is provided with a Venturi meter. After passing through the regulator house, also built of concrete, the water is then delivered by 64-in. and 72-in. concrete pipes to the pure water reservoir previously described. From this point it connects with a 6-ft. steel pipe which runs across the island to the tunnel leading under the bay and thence connecting to the main pumping station on the Toronto shore.

The office building has been erected entirely of concrete, and is two stories high with basement. By rubbing and finishing the walls of this building with carborundum brick a finish not unlike white marble was made possible.

REINFORCED CONCRETE STOREHOUSE FOR THE GENERAL ELECTRIC CO. AT SCHENECTADY, N.Y.

THE new storehouse of the General Electric Co. at Schenectady, N.Y., is 387 ft. long by 73 ft. wide, and five stories in height, and was built from the footings to the roof in 39 working days. In this time 6,100 cu. yds. of concrete and approximately

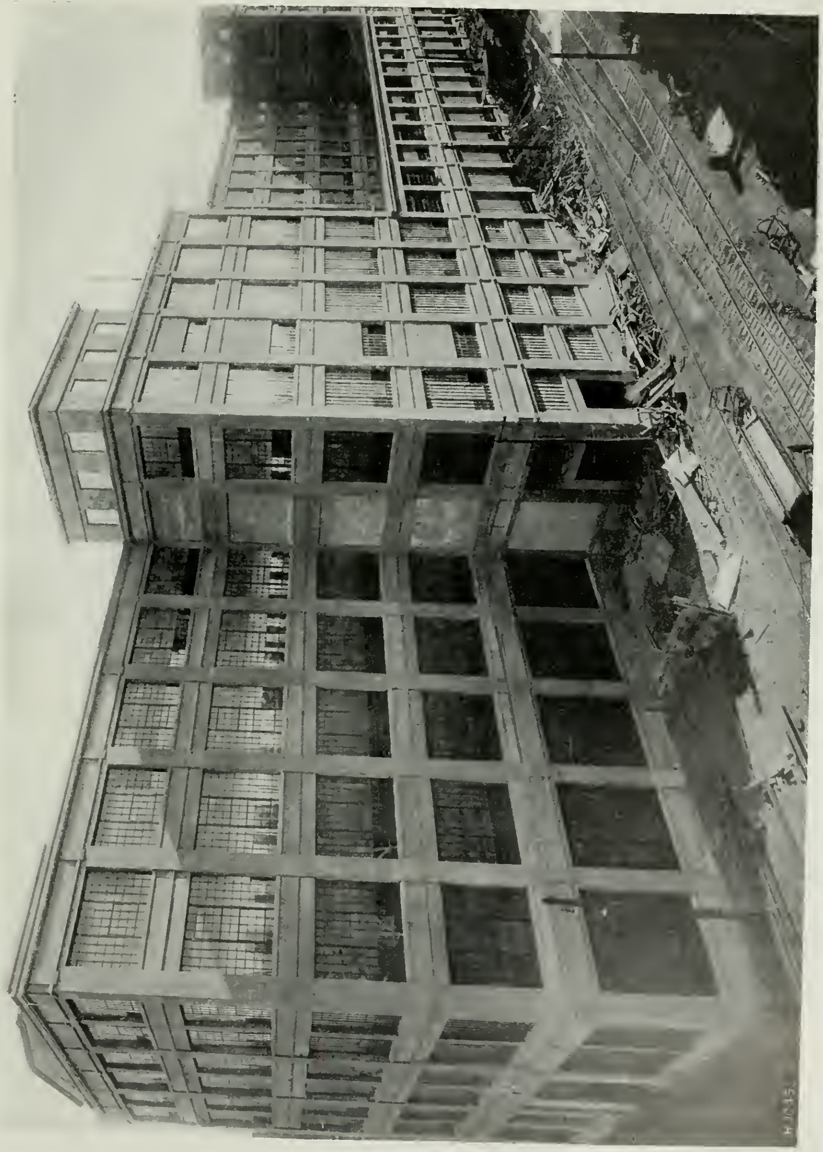


THE GENERAL ELECTRIC CO.'S PLANT, SCHENECTADY, N.Y.

1,180,000 lb. of reinforcing steel were placed. The following particulars of the work are taken from an article in our contemporary, *The Cement Age.*

In addition to the storehouse a new machine shop was built at the same time. This building is also five stories in height, and 83 ft. wide by 403 ft. long.

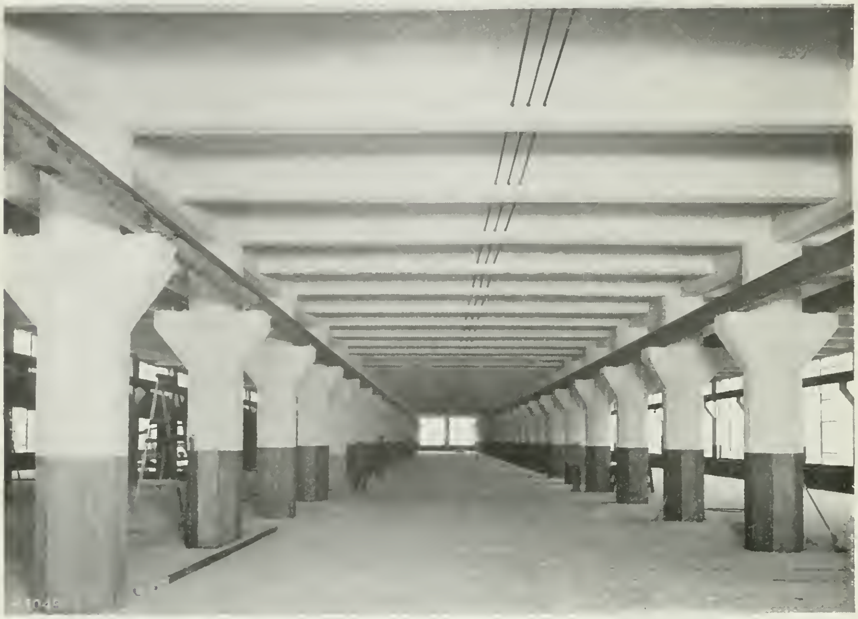
In both buildings reinforced concrete was used throughout. The second floor of the storage building was designed to carry 400 lb. to the sq. ft. live load, the third and fourth floors 300 lb., and the fifth floor and roof 200 lb. The machine shop was designed for a live load of 200 lb. to the sq. ft. throughout.



View of Machine Shop.
The General Electric Co.'s, Plant, Schenectady, N.Y.

In the machine shop building the bays are 27 ft. by 16 ft. The beams run across the building and are spaced 8 ft. on centres, framing into the columns and into the centre of the longitudinal girders alternately. The keynote in the design of these buildings is maximum economy in the amount of steel and concrete required. Cold twisted bars, the pitch of the twist being nine diameters of the rods, were used throughout. These rods have an elastic limit of 55,000 lb. per sq. in.

The concrete was mixed in the proportion of 1 part Portland cement, 2 parts sand, and 4 parts broken stone; in the columns a 1-2-3 mixture was used.



Interior of Second Floor, Machine Shop.
The General Electric Co.'s Plant, Schenectady, N.Y.

The exterior columns are of the Hennebique type, while the interior columns, in order to reduce their size, were hooped spirally. The largest column was 32 in. from face to face of the octagonal concrete form. The spiral was 28 in. in diameter.

The beams throughout the building were designed as T beams with uniform stems, thus making very few changes necessary in the forms and effecting a considerable saving in the cost of same.

The arrangement of the reinforcing in the beams and in the flange of the T section is very interesting. In the centre of each beam and near the top of the floor slab is located a loop bar extending the full length of the girder. This bar is supported on blocks resting on the floor forms, which are knocked out as the concreting progresses. From this loop bar all of the reinforcing bars in the beams are suspended by means of $\frac{3}{8}$-in. stirrups bent in the form of a W. One of the principal functions of the loop bar is to bind the stem of the beams into the slab, which it does by having every other slab bar cross underneath it, while the stirrups come up over it, thus giving a perfect distribution of the stresses from the stem up into the slab and out into the flanges. The main reinforcement bars in the bottom of the beams extend the full length of the span unbent. For the shearing stresses, the diagonal tension stresses, and the negative bending moments over the columns short bars are bent, the lower ends of these being hooked around a short cross-rod between the two lines of bottom reinforcing rods.

The work was executed in a remarkably short space of time owing to the effective organisation of the contractors, the Stone and Webster Engineering Corporation of Boston, who designed the work with a view to both speed and economy of erection.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"Graphical Reinforced Concrete Design." By John A. Davenport.

London: E. & F. N. Spon, Ltd., 57 Haymarket. Price 5/- net. 8vo. 48 pp. and 4 large sheets of diagrams.

Contents. — Part I.: Singly-reinforced Concrete Beams. Part II.: Doubly-reinforced Concrete Beams. Part III.: Columns.

There are now numerous diagrams for saving labour in reinforced concrete design on the market, and we fear it is overdone, though we welcome every one that is published, for all add something to the progress of science. Mr. Davenport has displayed much ingenuity and ability in evolving these diagrams to which the letterpress is explanatory, and doubtless in his hands they serve as a ready means of determining factors, but they are, we fear, somewhat too involved even yet for general use. The diagrams for eccentrically loaded columns are novel, and therefore most interesting to us. We do not quite like the ones for singly and doubly reinforced beams, as we think practical purposes could be more readily served by simpler, more direct diagrams. The diagram for shear members is novel also, but we think there must be a mistake on page 11 in the example given, for we get only 1 sq. in. of steel required at 12,000 lb. stress, or if the steel be stressed, to 16,000 lb. per sq. in. ·74 sq. in. will be required, whereas the author requires 1·49 sq. in. Possibly the error is in taking 24 ft., the whole length of the beam instead of half the length, as developing the reaction at the end which constitutes the maximum shear at the end, for Mr. Davenport gives just double what we require.

We are sorry this work was published just before the second R.I.B.A. report, because it is based on the first report and now seems a little out of date, while the notation employed is not that now favoured. However, we hope the author will have the opportunity of carefully revising it for a second edition, when we hope he will consider how he can simplify his diagrams, for which purpose we advise him to carefully study the other works on such subjects.

"Engineering Work in Public Buildings." By R. Owen Allsop.

London: E. & F. N. Spon, Ltd. 12/6 net.

Contents.—Introduction—The Lay-out of an Engineer's Department in Public Institutions—Vertical Boilers for Institutions—The Cornish Boiler Lancashire Boilers — The Dry Back-Boiler—The Water Tube Boiler for Institutions — Steam Heating — Hot Water Heating — Exhaust Steam Heating—Ventilation—Water Supply —Institutional Baths—The Institution Laundry—The Institution Kitchen—Lighting and Electrical Machinery.

We do not propose to criticise this book at length, as the subject-matter does not come within the purview of our journal, yet we think it will be found extremely useful and interesting by many of our readers, especially by municipal engineers and those connected with the power, heating and lighting of institutions. The articles originally appeared in the pages of "Surveying and the Civil Engineer," but they have been considerably revised and enlarged. The book is extremely well produced and printed.

"The Civil Engineers' Cost Book." By Major T. E. Coleman.

London: E. & F.N.Spon, Ltd., Haymarket. Price 5s. net.

This book, published in a very handy form, is a concise but comprehensive pocket price book compiled for the use of civil and municipal engineers and public works contractors. The purely engineering side of estimating is dealt with in detail in the book, which is a supplement to the author's "Approximate Estimates."

The first part deals with the cost of every class of civil engineering work and electric lighting; electric tramways and railways are also dealt with. The second portion gives the cost of engineering plant and machinery.

The book sets out to give preliminary estimates of the most diverse character at a moment's notice, or particulars as to the relative cost of two or more methods of executing some particular work, and the author has certainly achieved his object in a very useful way. The book should be of great assistance to the civil engineer and contractor in general practice.

CORRESPONDENCE.

Under this heading we invite correspondence.

March 24th, 1912.

Dear Sir,—Mr. Jackson, in his article on the Design of Reinforced Concrete Columns, has misinterpreted the recommendations of the Joint Committee on Reinforced Concrete with respect to the column formulæ.

It is true that the Committee proposed that the value of s should be limited to 32, but they did not intend to place any restriction upon the closeness of the pitch or the value of r.

It is distinctly stated on page 17 of their second report that "The spacing factor should not be taken at more than 32, even *if p is less than 0·2d.*, clearly indicating that p may be taken as a less value than 0·2d.

Mr. Jackson also states that the Committee recommended that the steel should not be stressed to a greater amount than 15,000 lb. per sq. in.

In the Committee's second report, page 18, it is recommended that "the stress on the metal reinforcement (*i.e., the value of m. c.*) should not exceed 0·5 of the yield point of the metal," meaning the stress of the yield point-resistance.

This means that for mild steel with a resistance at yield point of 32,000 lb. per sq. in. the allowable stress would be 16,000 lb. per sq. in., and that for steel with higher resistances at the yield point the allowable stress would be proportionately greater. Yours faithfully,

Charles F. Marsh.

The Editor, "Concrete and Constructional Engineering."

The Execution of Reinforced Concrete Work.

March 16th, 1912.

Sir,—Under the above heading in your issue for this month you mention a matter of very grave interest which has held my attention for some considerable time. I think that the progress of reinforced concrete construction in this country has been delayed quite ten years if not more through the negligence and incompetence of foremen.

But, sir, I disagree with you when you exonerate the contractors and specialists of all blame, for you cannot entirely condemn an ordinary building foreman for saying he can undertake to carry out a job in reinforced concrete when he cannot get one in his ordinary class of work.

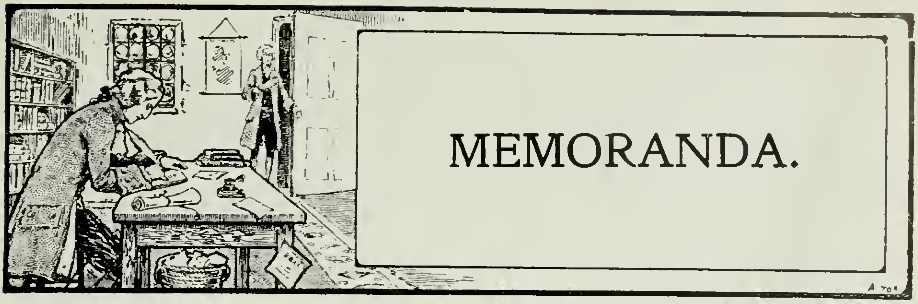
Now the contractor can get competent skilled foremen to carry out reinforced concrete work, for there are quite a few of them to be had at present with good sound experience if he will pay a reasonable price for ability. A man that is progressive and far-seeing enough to forego his past training and undertake the expense of learning a branch which is entirely new to him is above the ordinary type.

Contractors generally are very keen on getting reinforced concrete work and, therefore, often take it at an unreasonable price and resort to certain risks to make it pay. Well, a modern competent reinforced concrete foreman will not take some of these risks, and is often discharged in consequence.

Other instances are where contractors and architects have their old foremen and clerks of works on hand and try to do with them, but most of these men are of the old stock, who speak of reinforced concrete as a new-fangled idea and, because they have no knowledge of it, continually cry down its merits.

I think the remedy would be if the modern men who have the necessary knowledge supervised every job as clerk of works and were directly responsible to the engineers in charge of the reinforcements, and were paid a living wage to keep them above corruption. To do this, a register of these men should be kept, say, at the Concrete Institute, where they could sign on and off. If at any time one of these men misbehaved himself, when proven by an unprejudiced authority, he should be struck off the register and not employed in that capacity in future. This would ensure an experienced man on the site with the authority to keep things right. The engineering firm should in turn be directly responsible for this man.—Yours faithfully,

George Coe.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Tests on "Dentile" Floors.—We have received the following particulars of a successful test which was carried out on the 7th March, 1912, on the floors which have recently been constructed by Messrs. H. Arnold and Sons, Contractors, Doncaster, for the Midland Carriage and Wagon Works, Washwood Heath, Birmingham.

These floors have been constructed on the Dentile system, which is controlled by the Indented Bar and Concrete Engineering Co., Ltd., of Westminster. The feature of the floor is the use of hollow tiles, which have the effect of greatly lightening the floor itself, and at the same time making it sound-proof without the use of projecting beams either of steel or of reinforced concrete.

The tests in question were two in number. The first consisted of a square floor reinforced in both directions 17 ft. 10 in. by 16 ft. 7½ in. The working load for which this floor was designed was 100 lb. per sq. ft., but it was arranged to test the floor up to 1½ times this load—viz., 150 lb. per sq. ft. This load was considerably exceeded during the test. The material, by means of which the load was placed upon the floor, consisted of cement sacks, and in all exactly 20 tons were placed on this floor, which is giving a load of 152 lb. per sq. ft. The deflection was very carefully measured during the process of loading by means of a sliding vernier placed below the floor in the centre, and the deflection increased quite gradually during the progress of the test to 0·07 in. While the full test load was still on the floor the centre of the floor was exposed by the removal of a sack of cement, and five heavy blows with a piece of timber 18 ft. long (9 in. by 12 in.) lifted to a height of 2 ft. or 3 ft. and banged down upon the floor by the united efforts of five men, were given to it. The floor withstood this extremely severe test admirably, no cracks or other signs of damage being apparent.

The next test took place on a small, similar room 16 ft. 7½ in. by 13 ft. 10 in., in which the floor was constructed on the one-way system; that is to say, it was only reinforced in the smaller direction, there being no reinforcement whatever in the direction at right angles to this. The full load—which again was 1½ times the working load—consisted of 17·2 tons, but the deflection in this case only amounted to 0·01 in.

The extreme success with which these floors of both types withstood not only the heavy test load, but also the test for shock, affords the best possible proof that Dentile floors are suited in every way both to the carrying of heavy dead loads and to heavy falling loads with a minimum deflection and vibration. The floors are, of course, quite economical, and the surface afforded by the tile for plastering is exceptionally good. The absence of beams over large spans enables the ceiling treatment to be carried out with great ease.

A Novel Method of Pouring Concrete Bridge Floors.—In the construction of the main viaduct of the McAdoo Tunnel extension into Newark a very interesting method was used in pouring concrete bridge floors, which are trough-shaped and each 13 ft. wide, 11½ in. thick, and laid in sections 250 ft. long with expansion joints between.

The forms are built complete several hundred feet in advance of the concrete, and the reinforcing steel is set and tied by the force of the steel contractor before a run of concrete is started. Concrete is then placed by two travelling plants on the deck of the viaduct. One consists of a 7-ft. gauge car carrying a 1-yd. Ransome mixer and timber-feeding bins. It runs on the temporary track mentioned at one side of the slab which is being placed. This track has three rails to accommodate standard gauge and the 7-ft. gauge. A locomotive crane on the track accompanies the travelling plant, and supplies its bins from material cars standing on one of the outer freight tracks below. Concrete is delivered from the mixer to any part of the slab on the opposite half of the viaduct by a sectional chute, and is tamped in place by the concrete gang. Under this plan of operation 250 ft. of single track slab is placed in an 8-hour day, and at times two sections, or 500 ft., have been laid, the time required being fourteen hours. On a part of the work, in placing the second track-slab, a second Ransome mixing plant has been installed, consisting of a mixer on a flat car with the material cars coupled at the ends. This travelling plant moves over the completed first track and operates similarly to the other described.

There is every indication that this method of pouring bridge floors is thoroughly economical and efficient, and these two travelling mixing plants are well worth the study of contractors anticipating this kind of work.

A Reinforced Concrete Pavement at Plymouth, Wis., U.S.A.—During the summer of 1910, says *Engineering Record*, a reinforced concrete pavement was laid in Plymouth, Wis., having three features of construction which are unusual in the laying of a pavement of this kind. These new features of construction are the use of "Pecky" cypress for expansion joints, a reinforcement of the concrete with a woven wire mesh and a new form of rough surface. In place of the usual asphaltum or tar expansion joints, 1 in. by 8 in. cypress boards were used along each gutter and every 4 ft. across the street, and on those streets where street car tracks had been laid a joint was made on each side of the track at the end of the ties. These boards were used as forms for the outside of the cement gutters and as a template in forming the crown of the street, thus saving the use of considerable timber, as well as time in placing and removing the same.

The woven wire mesh reinforcement was placed directly on the base concrete so as to lie between the surface and the base. By using this mesh and placing the expansion joints every 40 ft. the pavement was cut up into monolithic squares of 40 ft.

The surface finish coat consisted of crushed granite chips ranging in size from $\frac{1}{4}$ to $\frac{3}{4}$ in. in diameter, granite screenings from $\frac{1}{4}$ in. down to dust, and Portland cement mixed in that proportion which gives the densest mixture. After this mortar was spread upon the mesh and concrete base it was levelled off with a street screed and then trowelled smooth like a sidewalk. While the material was still soft, granite chips, ranging from $\frac{1}{4}$ to $\frac{3}{4}$ in., were cast by hand or shovel over the surface of the pavement, giving to the entire area a roughened appearance. Some of the stones disappeared entirely into the soft cement mortar, leaving small depressions in the pavement, while others sank in only part way. This made a surface which, while generally level, was still rough, but with variations so small that no annoyance is felt in riding over it; nor do the wheels and shoe calks, which drop into the deep grooves, cause unnecessary wear.

Outside of the special features mentioned, the pavement was laid according to standard concrete specifications. The base was made 5 in. thick and the surface coat $1\frac{1}{2}$ in. thick. This pavement is now over one year old, has passed through one winter, and there is not the sign of a crack or a flaw in it, even along the street car tracks.

The use of the woven wire mesh is held to have demonstrated the fact that it is possible to construct a concrete pavement so that it will not crack along the centre of the street. This new form of surface is not slippery and does not wear perceptibly, according to reports from engineers who have examined it.

Concrete River Wall, Burnley.—The contract for the new concrete river wall at the Altham Sewage Works for the Corporation of Burnley has been let to Messrs. M. and C. W. Heap, of Burnley, and is to be carried out under the supervision of Mr. C. H. Pickles, M.Inst.C.E., the Borough Engineer to the Burnley Corporation.

Reinforced Concrete Retaining Wall.—Mr. E. B. Martin, M.Inst.C.E., the Borough Engineer of Rotherham, has recently been asking for estimates for the construction of a retaining wall in reinforced concrete alongside the River Don, with the object of making a clearer waterway and resisting the ever-increasing erosion of the bank and the silting up of the river.

TRADE NOTICES.

Some New Specialities of the (U.K.) Winget Company's.—We are glad to note that the (U.K.) "Winget" Concrete Machine Co., Ltd., of Newcastle-on-Tyne, have added to the list of goods made by them the "Ross" Concrete Spade and the "Andrews" Concrete Tamper—two implements which should prove valuable adjuncts to every concretor. These tools do away with the necessity of using fine facing material when concreting, thus considerably reducing costs.

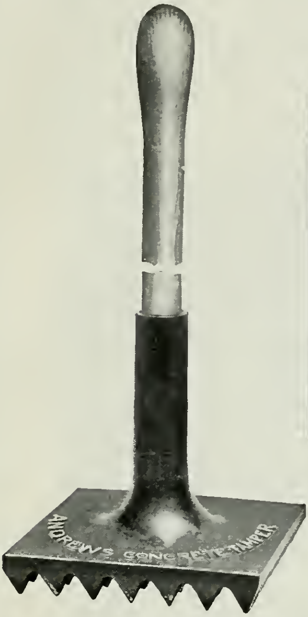


FIG. 1. THE "ANDREWS" TAMPER.

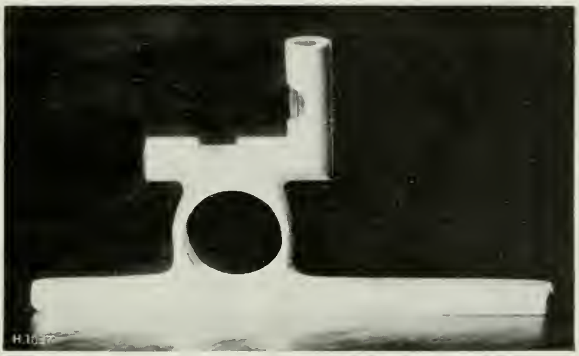


FIG. 2. THE "KOKOMO" LEVEL.

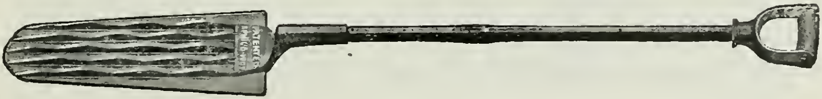


FIG. 3. THE "ROSS" SPADE.

The "Ross" Spade, illustrated in *Fig.* 3, is used where shuttering is employed, the concrete being rammed with the spade close to the shuttering, thus forcing back the coarser aggregate, and allowing the fine material to settle up to face of work, obviating the necessity of "cement rendering" or otherwise facing concrete walls.

The "Andrews" Tamper, shown in *Fig.* 1, is used as a rammer when laying concrete paths, roads, etc., *in situ*, the tamper achieving the same effect of bringing the fine particles of concrete to the surface, which may be "floated off" at once, thus a finishing coat for "floating off" is not required.

Nor should we omit to mention that this Company have also placed on the market an exceedingly useful level, viz., the "Kokomo" Finger Ring Level, an illustration of which we give in *Fig.* 2, and which may be utilised either for levelling or plumbing.

We would strongly advise every builder and contractor to get particulars of these indispensable tools, which with ordinary care will last a lifetime.

Lock Woven Mesh.—The business of the lock woven mesh system of reinforcement has been taken over by Messrs. George F. West & Co., of Caxton House, Westminster.

THE BRITISH STEEL PILING COMPANY

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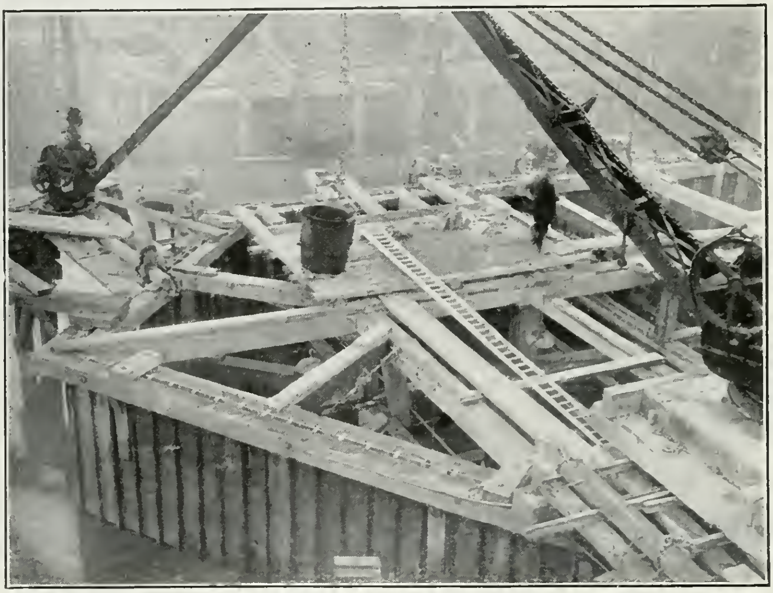


Illustration shows one of three cofferdams constructed with our steel piles at L.C.C. Reconstruction, Deptford Creek Bridge.

Universal Joist Steel Sheet Piling for Cofferdams, Retaining Walls, etc. etc.

Used by the BRITISH AND COLONIAL GOVERNMENTS, FRENCH GOVERNMENT, AND LEADING CONTRACTORS AT HOME AND ABROAD.

Latest Types of Pile-Driving Equipments

In a handbook dealing with this system, which has recently been issued by the proprietors, full particulars are given of this form of reinforcement, together with illustrations of the different structures in which it has been applied. The employment of "lock woven mesh" is particularly suitable for the construction of roofs, floors, foundation rafts, walls, sewers, culverts, etc.

As the name implies, the reinforcement in this system is a wire-woven mesh which is securely locked together at the intersections. The mesh is formed of main or "tension" wires and secondary or "transverse" wires. The former are spaced from 3 in. to 6 in. apart, while the latter are spaced about twice the distance apart of the tension wires. The Standard widths of the fabric are 4 ft. and 5 ft., and the material is stocked in these widths in rolls of 100 ft. to 200 ft., but there is no limit to the length in which the fabric can be supplied.

Lock women mesh is being used by H. M. Office of Works as reinforcement for the covering of the reservoirs in connection with the Windsor Castle Water Supply. It is also being used for reinforced concrete bunkers at Burslem; concrete roof at Consett; reinforced concrete floors at Students' Hostel, Dublin, and other smaller contracts.

The "Sasso" Reinforced Concrete Street Kerb.— Kerbs of natural or artificial stone, when used for pavements, have the disadvantage that owing to unequal settlement they may be tilted or displaced relatively to one another. This difficulty has led to the use of larger and heavier stones. The provision of a concrete foundation is costly, and is not always a sufficient safeguard.

Fig. 1

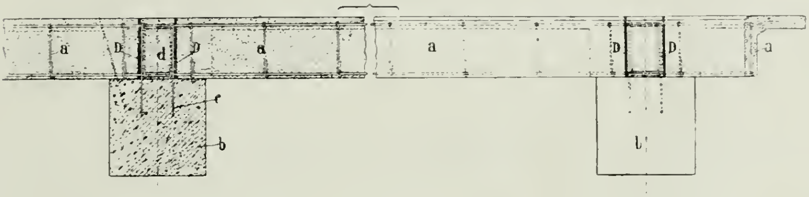


Fig. 2

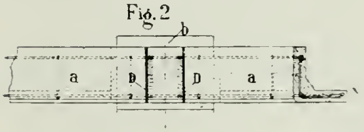
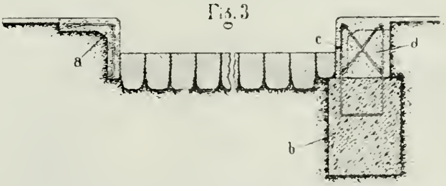


Fig. 3



The "Sasso" Reinforced Concrete Kerb is intended to overcome these difficulties, and consists of a continuous reinforced beam "a" carried at ends by the supports "b." In *Fig.* 3, on this page, the section on the left is cut through the kerb, and that on the right through one of the supports. A lean ballast concrete may be used for the latter, whilst sharp-edged fragments of basalt, or other hard stone, broken small, are used for the concrete in the beams. The great reduction in section of the kerb proper, as against stone, causes economy of material, and irregularities of level due to tilting or settling are entirely prevented. The wear is less than that of natural stone, and is also more uniform.

A similar construction may be employed for the edges of paths in parks and gardens, the protecting kerb around monuments, and the foundations of light structures, such as verandahs, etc. Street kerbs on this system have been used in different parts of Roumania, and have been found very satisfactory.

Full particulars can be obtained on application to Mr. L. F. Szilagyi, 13, St. Helen's Place, E.C.

"Pudlo" Waterproofing.—A material which has recently been put on the market for waterproofing brickwork, stonework, and concrete is known as "Pudlo" and is manufactured by Messrs. Kerner-Greenwood and Co., of King's Lynn.

"Pudlo" is a fine white powder which is mixed with neat cement in varying proportions, according to the purpose for which it is required. It is claimed that when so mixed it absolutely prevents the penetration of dampness and moisture. It not only protects the exterior of the structure, but in the event of the surface being accidentally chipped it still keeps the water from penetrating.

The proprietors have just published a booklet which gives full particulars of the method of using Pudlo and of various experiments which have been carried out to prove its efficacy.

The booklet will be sent to anyone applying to Messrs. Kerner-Greenwood and Co., King's Lynn.

The British Reinforced Concrete Engineering Co. inform us that Mr. A. W. Storey, of the Barb Engineering Co., Manchester, has joined the Board of the British Reinforced Concrete Engineering Company.

Messrs. D. G. Somerville & Co. inform us that Mr. R. N. Stroyer, late principal Engineer to Messrs. Christiania and Nielsen, the well-known firm of Swedish reinforced concrete designers and erectors, has recently been appointed to the post of Chief Engineer in the firm of D. G. Somerville and Co.

CHANGE OF ADDRESS.

Messrs. Richard Johnson, Clapham & Morris, Ltd., inform us that, owing to the increase of their business in London, their agent, Mr. H. M. Kelk, has moved his offices from Holborn Hall. His address now is Broad Street House, New Broad Street, London, E.C.

CONTRACTS.

Messrs. D. G. Somerville & Co., have recently secured a large contract for the erection of an extensive T jetty on the Thames, which is to be entirely constructed of reinforced concrete. The jetty, which is to carry three large electric cranes, will be over 230 ft. long.

Among other contracts which this firm have in hand at present are:—Water tower at Bembridge; bank at Banstead; building in Tottenham Court Road; water tanks at Aylsham, Norfolk; shops and flats, High Street, Clapham; warehouse, Wapping; two office blocks in Tudor Street, E.C.; two blocks of flats, Adelphi; floors in De Keyser's Hotel; floors and galleries, Victoria Station; reinforced concrete sleepers for the India State Railways; sanatorium in Bournemouth.

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Three feet long to Nine feet long.

Breeze Partitions
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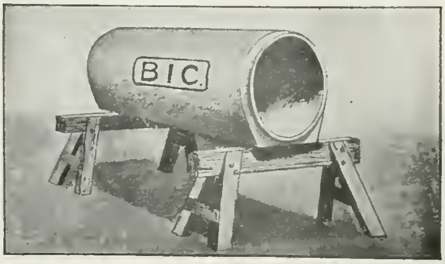
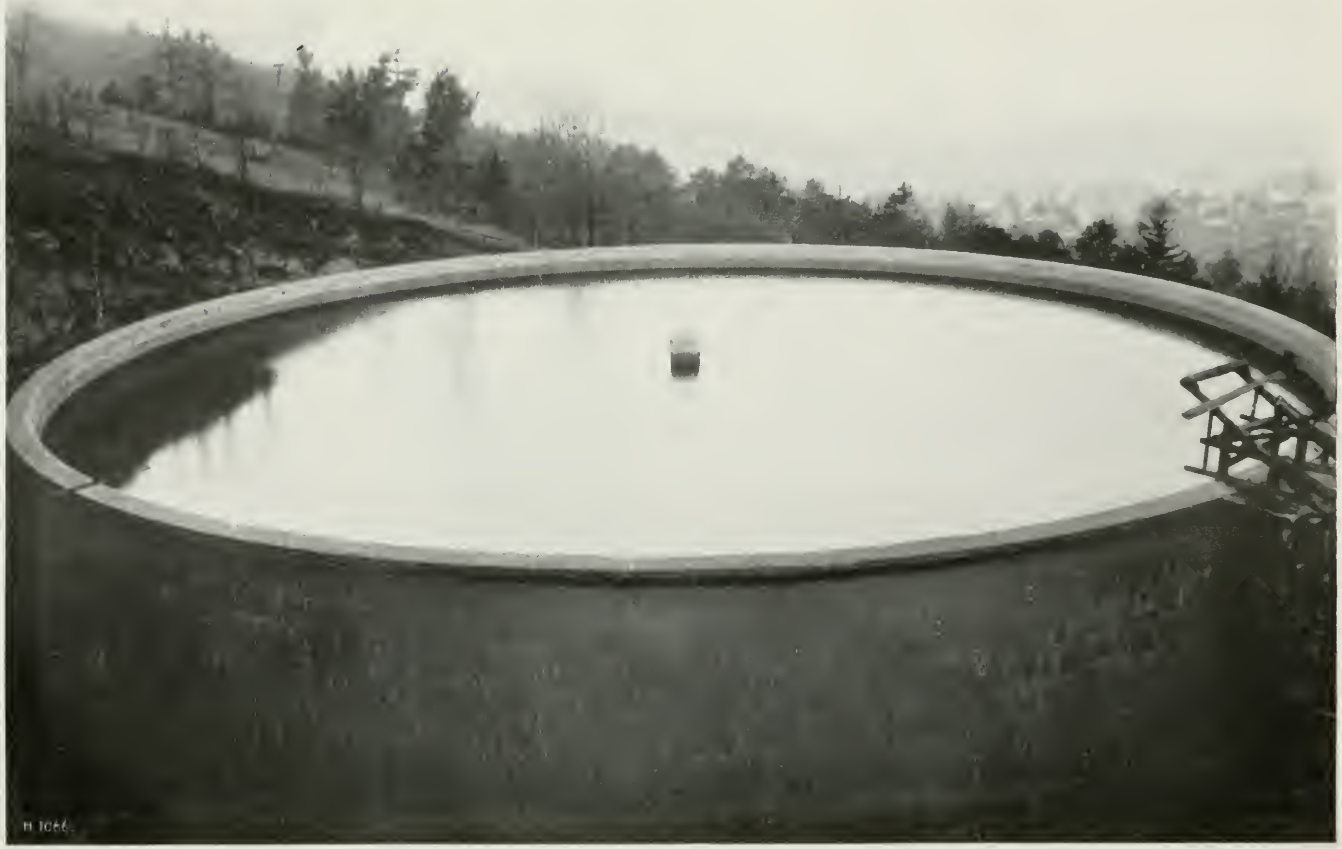


Porous Pipes
Concrete Beams
Hollow Floors

SPECIALITY:—INVENTORS' DESIGNS CARRIED OUT TO ORDER.



For description see page 387.

Five-Million-Gallon Reinforced Concrete Reservoir at Ashville, N.C., U.S.A.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 5. LONDON, MAY, 1912.

EDITORIAL NOTES.

THE EXCESSIVE SPECIFICATION.

THERE are certain everyday matters in the design of reinforced concrete which get little talked about by specialists, because from long experience with them they have become commonplace; and yet they ought to be continually emphasised and thrashed out with their professional *clientèle* because it is a matter in which the latter have to decide, and which leads to continual friction and want of uniformity. That is, in the question of loads on floors and the factor of safety.

In talking of factors of safety there are various interpretations that can be made, and the specialist designers use their discretion. Almost all architects and engineers are convinced of the necessity of asking for a factor of safety of four, and practically never ask for less. Now, such a definition is extremely loose because if a factor of four is required on the ultimate breaking load, it is quite a different thing from designing for a working stress which is one-fourth of the ultimate stress which the material is capable of sustaining. In dealing with a stipulation that the factor of safety is to be four, there may be a considerable variation on the part of the designer according to his interpretation of the wishes of the client. However, most specialist designers are fairly uniform in this respect, because they work upon certain lines which they have adopted to satisfy themselves, so there is ordinary common practice among them.

On the question of loads on floors, however, it is a common fault in engineering practice for the super loads to be grossly over-estimated. Now, when a specialist is " up against " a requirement that the super load on a schoolroom floor shall be taken as 3 cwt. a sq. ft., and he knows that $\frac{1}{2}$ cwt. is as much as the floor will be called upon to sustain, or when, say, 2 cwt. is specified for a hospital ward floor and the load in practice would not exceed 25 lb. per sq. ft., he naturally looks to see if this absurd requirement can be dodged, withdrawn or reduced in effectiveness. He knows that it is distinctly absurd to adhere to such a requirement, and if he does not allow for a much smaller load he will be almost sure to lose the job to some competitor, and he knows also that the architect or engineer would not be willing to get the building owner to pay for a floor capable of sustaining the load he specified. Of course, where it is stated that the floor will be tested with the specified load, then the floor must be built stronger than it otherwise would be, but even then, knowing the cost of testing, and that it is unlikely that a floor would be so loaded subsequently in practice, the specialist seeks means of reducing the altogether preposterous specification.

Now, professional men are not always open to reason, and if a specialist designer comes along and presses the point, and emphasises how absurd and how ignorant it is to put forward such a requirement, he will not find it to his advantage. Of course, it all depends upon how the representation is made, but the suggestion of any reduction in the requirement is not always welcome, and the specialist's suggestions are looked upon with suspicion. If he finds it easy to get the specification modified, that is the obvious course to follow, but if any difficulty is met the designer naturally looks for other ways of escape, and a spirit of generosity enters into his estimation of weights and spans. He under estimates, in all probability, the dead load of the floor; he takes the minimum span that he can possibly find arguments to support, and he uses his ingenuity generally to satisfy the client and yet produce a job that, even then, is possibly very much stronger and more costly than it ought to be.

It depends, therefore, very much on the ingenuity of the specialist as to whether he gets a job or not in competition, and consequently there are wide variations, just depending on how each specialist looks at the matter. It might be argued that it is very reprehensible practice not to give the client what he asks, and apparently what he wishes, but it seems quite legitimate for the specialist to use his ingenuity to devise a proper and adequate scheme, and persuade the client to adopt, or even carry out unknown to his client, what he knows to be a thoroughly sufficient scheme.

There are cases, of course, where the client may have in his mind some possible use of the building for a purpose which is not apparent to the specialist, and then it may be bad for the specialist to exercise his own opinions, but it would be far better if the architect or engineer were to consult a specialist or two before preparing his specification. We do not believe, on the other hand, that it is desirable to leave all to the specialists, because they have not all the information available, and they are, perhaps, submitting a scheme for a job miles away, without any knowledge of the site or local conditions, and when it is left too open for them to interpret what they will it seems that the man who takes the biggest risk gets the job.

Specialists would prefer to work to a sound but reasonable specification. In laying down such a specification there should be a really clear understanding of what is a right and proper load to require, and it would be best if the specification provided for some set of rules to be worked to, such as, say, the rules put forward by the R.I.B.A. Joint Committee. The competition would then be fair, and on the same basis, and it would soon be found that in such cases the variations would not be so wide, and all designs would be very similar to each other; but in using such a standard it would be worse than the old practice if the loads were to be overestimated as they have been in the past.

In the London County Council regulations for steel-framed and reinforced concrete structures the floor-loads of various classes of buildings are specified, and these are on a proper conservative basis, and it would be well if they were adopted as a basis throughout the country, being modified by reduction, and very seldom by any increase. Yet one finds that in very many cases floor-loads are very much heavier than these loads for which one is officially required to design for London.

One notices a tendency among specialists, where such excessive loads are required, to take the spans as being from the inner edges of supports, and often, too, allowances are made for fixity at supports, so that by dint of such reduction the bending moment is reduced to somewhere approaching what it would be if the floor were designed for a proper super load in the first instance. In continuous construction the smallest possible bending moment is taken. Such dodges, however, are to be regretted, as all dodges are, in that they obscure the point at issue, and they lead to the repetition of erroneous and foolish practices, which may sometimes result in ill-feeling and loss to someone.

One can understand professional men setting down in their specification what they will require, and insisting upon getting it, because there is a natural tendency on the part of all contractors and others to put the minimum interpretation upon everything, so that by contrast the strict letter of the law is insisted upon. However, every specification must be interpreted with reason, and we are sure that many architects and engineers would be greatly surprised if they were taken always at their word, for then the cost of work would go up enormously.

So in reinforced concrete work, if the loads specified are very severe, the architect would be much surprised at the excessive cost of such work. He has probably had similar work carried out before under a free interpretation at a certain cost; he would not think of paying more again, and he has probably based his estimate on such low costs being obtained again.

We, therefore, see that one should be on the watch against excessive provisions in the specifications, and that the contractor or specialist designer is not to be dismissed as a humbug and a scoundrel because he takes liberties with an architect's excessive requirements. The architect should be reasonable and intelligent, and not act in an inconsistent and arbitrary manner; if he does, then he has got himself to blame for any troubles that arise.

WORKING STRESSES v. ULTIMATE STRENGTHS.

We have referred above to the question of designing for working stresses in the materials, or for ultimate strengths of the structural members. The tendency seems to be all in the direction of limiting the working stress, but we do not think this is always quite the best policy. It may be legitimate in many cases, but we suggest that it is not quite a scientific way of regarding engineering design. The practice has arisen as an outcome of steel-work design in which, whether we design for working stresses or use a reduction factor on ultimate strengths, we obtain practically identical results. This we see clearly if we take the simple case of a rolled steel joist. The working stress in the flanges is about a fourth of the ultimate stress that mild steel is capable of standing. Recent experiment and theory has appeared to prove that steel fails by maximum shear stress, and consequently, if we take the maximum fibre stress in tension and compression as some proportion of the maximum shear stress, we are fairly correct. However, the compressive resistance of steel is less than the tensile resistance, for reasons which are plain in higher theory, and, therefore, in specifying a safe working stress as determined from the tensile resistance of steel, we are, in a sense, adopting a wrong procedure, but in the case of a rolled steel joist we are not very far wrong, for the ultimate load which the joist will sustain is generally about four to five times the safe load that would be obtained from reckoning upon the basis of the working fibre stress.

Now, with a steel joist the variation in size and shape of the section is negligible; we obtain almost exactly what we design for. But it has been argued that as the variation in shape and form of steelwork is so small that the variations in practice for which we require a factor of safety are in respect to (1) the quality of the material; (2) errors of calculation; (3) secondary stresses not allowed for in the design; (4) mistakes in the carrying out; and (5) inferior workmanship.

Variation in the quality of the material seems to have struck engineers as an important factor for consideration in relation to the yield point, loading beyond which causes permanent deformation of the member, the yield point being about half the ultimate strength, from which the reduction factor of 4 on the ultimate becomes practically only a reduction factor of 2 on the elastic limit. It is thought that the variation in quality of the material was inclined to reduce this factor of 2 considerably, and, apparently, in the minds of designers, this is almost more important in steelwork than the inaccuracies in design and execution, so that they specify a safe working stress which shall be well within the elastic limit of a poor quality material. Supposing we take ordinary mild steel, which has an ultimate tensile strength of 28 to 32 tons, we know, as often found from tests, that the steel in practice may be reduced in quality so that it stands a test of only 22 tons to 26 tons. Taking the lower limit—namely, 22 tons—and reducing this by 2, we obtain an ultimate limit of 11 tons. We therefore see that in stipulating that the maximum safe working fibre stress shall be 7½ tons we have a factor of only about 1½, so that, due to poor-quality material, we have reduced our factor from 2 to 1½, or 25 per cent. It is not considered that the other reductions, due to inaccuracies in design and execution, will amount to as much as 25 per cent.; if so, we should have no factor at all against permanent deformation.

In reinforced concrete work it is a very different state of affairs. Our steel may certainly vary in quality as much as in steelwork construction, but that does not mean, therefore, that the same factor of 4 on the ultimate is applicable, because, for one thing, it may be that the risks of inaccuracies in design and workmanship are more, and we must also bear in mind that we are dealing not solely with steel but another material, concrete, in which the quality is much more variable than steel. Really, what our factor of safety is intended to do is to safeguard us against all risks, and we should, of course, limit our stresses on the one hand by reason of inequalities in the materials, and, on the other hand, we must put limits to safeguard us in case of errors in design and workmanship.

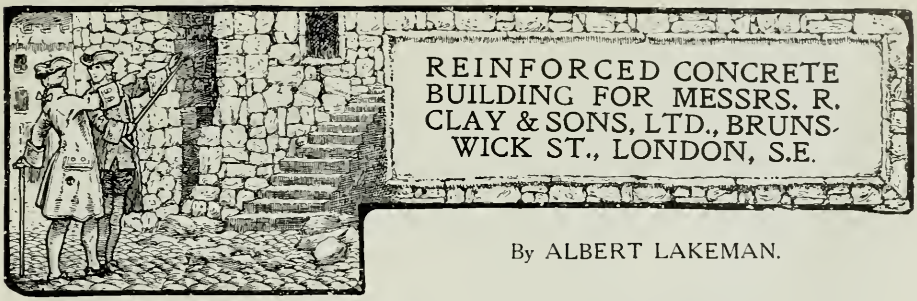
Supposing we take for example a reinforced concrete beam, we see how it is easily possible for the steel to be placed some distance out of its proper position. Supposing we have designed the beam to be 24 in. in effective depth, and that the steel gets higher up than its proper place in the beam by 2 in., our depth becomes reduced to 22 in. Roughly, we may say, then, that the strength of the beam is reduced by the squares of these quantities—namely, 576 to 484, or 16 per cent. If, on the other hand, the beam should be, say, 18 in. deep, and we get as much as 2 in. out of place, then it would be in the ratio of 324 to 256—namely, 21 per cent. We see, therefore, that we may get a very considerable difference due to the change of shape or size of a reinforced concrete beam; on the other hand, we can set against this the fact that we are using numerous small sections of steel, and consequently the risk of flaws and other reductions in quality are reduced, while, also, there is less risk of corrosion, which has to be considered in steelwork construction as regards the reduction in quantity, so that such risks of displacement are, to some extent, offset. However, the designer must take into consideration such risks. Then, again, in connections and other details there are more risks than with steelwork construction.

As regards the concrete, here, of course, is the chief point of variation in quality in reinforced concrete. The material is not manufactured under proper supervision as is steel. The concrete has to be done on the job, and there are the risks of (*a*) the cement being injured by storage; (*b*) bad mixing; (*c*) weather conditions; and (*d*) various other matters which tend to reduce the quality of the concrete from the highest value obtainable. Consequently, it has been customary to take a big factor of reduction for the stress in the concrete, and legitimately so. The stress is usually specified at about $\frac{1}{6}$ of the ultimate obtainable with very good quality workmanship at three months, but as the strength of concrete goes on increasing with age, and may increase this value by more than 25 per cent., the factor is very often about 8. However, there is another point to be considered in reinforced concrete design which distinguishes it completely from steelwork. Steelwork has a uniform modulus of elasticity practically up to the yield point, which is half the ultimate, and the yield point is raised by over-strain; but in concrete there is no such uniform modulus of elasticity, which becomes reduced the higher the stress, so that in designing reinforced concrete beams and columns for their ultimate resistance we must adopt different formulæ to what we do if we are designing at working stresses, because the distribution of stress is different. There is in concrete no well-marked yield point, and we may say, roughly, that a point somewhat corresponding to it must be placed at about three-quarters of the ultimate strength, although in small loads there is a certain amount of permanent deformation, which, however, becomes less and less on reloading.

Now this constitutes a very distinct difference, and requires an alteration in the outlook on the whole question of the use of a factor of safety. As we said before, by reducing the stress in steelwork to one-fourth of the ultimate strength, we have not so very much difference in working with a factor of 4 on the ultimate strength of a steel member—that is to say, we find a steel member will withstand about four to five times the working load at the point of failure. Reinforced concrete is, however, quite different. Due to the change in the law of stress distribution, we find that whereas one beam may, when tested, sustain four times the working load for which it was designed, other beams of different dimensions may sustain twelve and more times the working load. Now steelwork has led us to think that the obvious course should be to reduce the working stress in the material to a safe limit, because the variations in workmanship were not so much as variations in the material. But, after all, in steelwork we really had in mind the fact that the structural member must be able to sustain the working load with a margin of safety. That is equally true of reinforced concrete, and we see, working with limits imposed as to the stresses in concrete, we may obtain beams with a very variable factor as regards ultimate strength.

It is a matter for very serious consideration whether it would not be better to discard the basis of designing for working stresses, and design with a factor on the ultimate strength as well. Thus, suppose we say we take the maximum reduction in ultimate resistance of the materials, and next consider the maximum variation in disposition of reinforcement which would reduce the ultimate strength. Then if we have a concrete which has, say, an ultimate compressive resistance, in cubes made with fair workmanship, of 6,000 lb. per sq. in. at three months—deduct 50 per cent. for inferior workmanship and materials and secondary stresses—take three-quarters of this, we obtain 2,250 lb. as the yield point. We now require to design a beam with this as the maximum stress in the concrete. We should take the equivalent modulus of elasticity of such material. The stress in the steel we will suppose capable of withstanding a maximum stress of, say, 23,000 lb. We then, say, reduce the effective depth 20 per cent. If with these limits we make the beam capable of sustaining a load which is twice the working load, we should feel we had safeguarded ourselves in all respects in design and execution.

This is what might be more legitimately called designing with a factor on the safe ultimate, in distinction to designing for working stresses, and it is open to debate as to which is the best method for adoption at the present time. We think that an engineer in practice should not bind himself to either method, but use his discretion in the matter. The difference in the methods of designing should, however, be thoroughly recognised and borne in mind in the design of any work, because, at certain points, the two methods differ widely from each other, and at these points it seems to us that designing for safe ultimate stresses is more logical and better practice than the other, although it may result in a reduction in the sizes of members. We can, however, by this method feel quite safe because we can see clearly, at every step, how every risk of variation and reduction in strength is met, and if we feel that in the particular circumstances of any job a bigger allowance should be made for such contingencies, we can easily make it. We proceed more clearly with our eyes open by this method than by the working stress method, and, moreover, in many cases the safe ultimate method gives smaller beams. Engineering is, after all, a matter of utilising knowledge to the best advantage for the benefit of the community generally. If, therefore, we can devise methods whereby we can obtain proper safety, which shall also be more efficient and consequently more truly economical than former methods, we ought to adopt them. Then shall the engineer have fulfilled his function as a man of science.



REINFORCED CONCRETE BUILDING FOR MESSRS. R. CLAY & SONS, LTD., BRUNSWICK ST., LONDON, S.E.

By ALBERT LAKEMAN.

The building described below is of special interest as it is of reinforced concrete construction throughout and is not faced externally with any other material.—Ed.

This building, which has been erected for Messrs. R. Clay & Sons, Ltd., is interesting, as it has been constructed entirely of reinforced concrete, and. unlike many buildings of a similar character, it is not faced externally with brickwork or stone, which disguises the constructional members, but has a concrete treatment throughout, and this fact renders it worthy of special attention.

The system employed was the "Kahn," designed by the Trussed Concrete Steel Co., Ltd., and the result is successful, illustrating as it does the possibilities of a concrete elevation. The architect was Mr. G. F. Collinson, F.R.I.B.A.

The building has a frontage of about 260 ft. to Brunswick Street, while the total area of the site is over 16,000 sq. ft., and there are five floors above the basement.

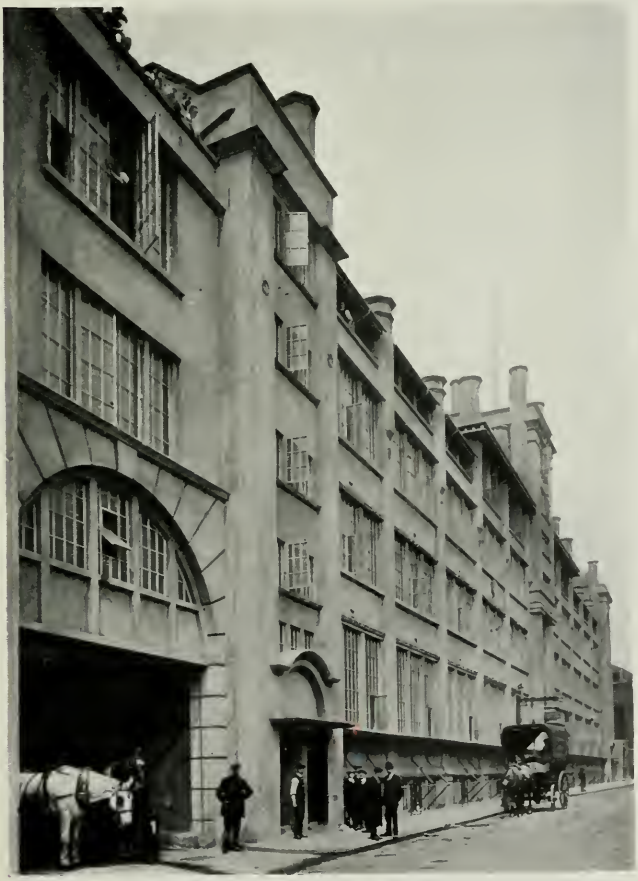


Fig. 1. View of Finished Building.
Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.

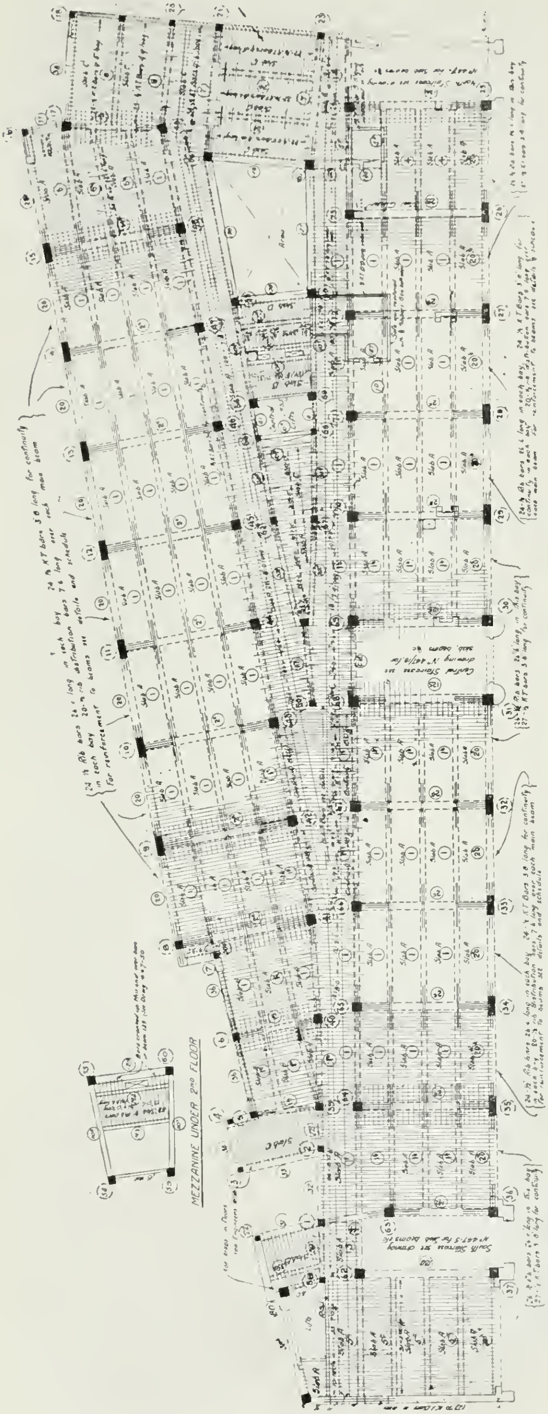


Fig. 2. Plan showing Second Floor Construction.

Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.

The foundations under the columns varied considerably, according to the loads to be carried, and the proximity of one column to another in the interior of the building rendered it necessary in some cases to put one raft to carry two or more columns. The foundations to the main front columns vary from 10 ft. 3 in. to 11 ft. square, and, generally speaking, consist of a slab varying from 14 in. to 26 in. thick reinforced with 1-in. Rib bars at 6½-in. centres and carried by cantilevers 60 in. deep and 30 in. wide across the full width immediately under the column, these beams being reinforced with four 1-in. and four ¾-in. Rib bars.

These front columns are spaced at about 20-ft. centres, dividing the elevation up into bays, and while consisting of ordinary reinforced columns in the basement they are spread at the top by means of a strongly reinforced bracket to carry twin columns on the ground floor, the latter each having an internal core diameter of 17 in. and being reinforced with six ⅞-in. vertical Rib bars and ½-in. hooping at 2-in. pitch. Above the ground floor storey the columns are practically replaced by piers, which are reinforced with a lattice of rib bars spaced at about 9-in. centres. The windows

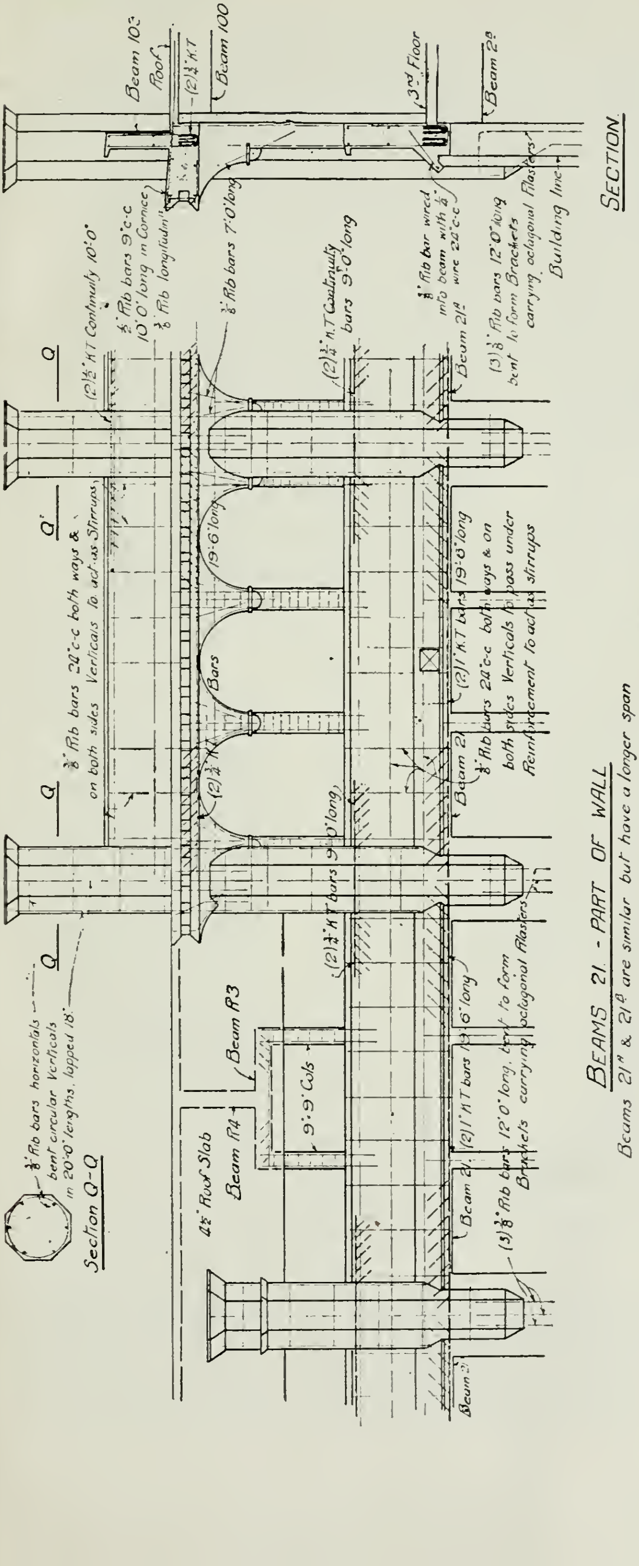

Section Q-Q
3/8" Rib bars horizontals bent circular Verticals in 20'-0" lengths, lapped 18"
4½" Roof Slab
Beam R4
Beam R3
9'-9" Cols
Q
Q
Q'
Q
3/8" Rib bars 24"c-c both ways & on both sides Verticals to act as Stirrups
(2)1½" K.T Continuity 10'-0"
3/8" Rib bars 9"c-c 10'-0" long in Cornice
3/8" Rib longitudinl
3/8" Rib bars 7'-0" long
19'-6" long
Bars
(2)1½" K
(2)1½" K.T bars 9'-0" long
(2)1½" K.T Continuity bars 9'-0" long
Beam 103
Roof
(2)1½" K.T
Beam 100
3rd Floor
Beam 2B
SECTION
3/8" Rib bar wired into beam with 1/8" wire 24"c-c
Beam 21B
(3) 3/8" Rib bars 12'-0" long bent to form Brackets carrying octagonal Pilasters
Building line
(2) 1" K.T bars 19'-6" long
3/8" Rib bars 24"c-c both ways & on both sides Verticals to pass under Reinforcement to act as stirrups
Beam 2
Beam 21
(2) 1" K.T bars 19'-6" long
(3) 3/8" Rib bars 12'-0" long, bent to form Brackets carrying octagonal Pilasters
Beam 2
Beam 21
BEAMS 21 - PART OF WALL
Beams 21A & 21B are similar but have a longer span

Fig. 3. Detail of Projecting Features and Turrets.
Reinforced Concrete Building for Messrs R. Clay & Sons, Ltd.

between these piers are formed into three lights having reinforced concrete mullions, while beams are formed at the heads to span between the piers. These beams vary in thickness from 15 in. to 18 in. and they are reinforced with two ¾-in. Kahn bars in the tensional area and two Rib bars in the compression area. Continuity of construction is ensured by the introduction of two ¾-in. Kahn bars in the upper surface where the beams pass over the piers, and resistance to shear is provided for by vertical rib bars, which are bent round the upper and lower reinforcement, thus binding the whole together. A projecting feature is formed in the centre of the façade at the third floor level, and this is carried by reinforced beams 12 in. wide and 5 ft. deep, while small octagonal turrets are introduced formed of solid concrete and reinforced with six ⅜-in. vertical Rib bars and similar horizontal bars at 24-in. centres bent to a circular form.

The main cornice

is also constructed in concrete and is reinforced with ½-in. bars at 9-in. centres and ⅜-in. bars placed longitudinally, while the parapet above is formed as a reinforced beam. It is not too much to say that every feature in the façade forcement to the slabs consisted of ½-in. Rib bars at 9-in. centres, and these were is quite unusual in the present-day methods of design and construction, when

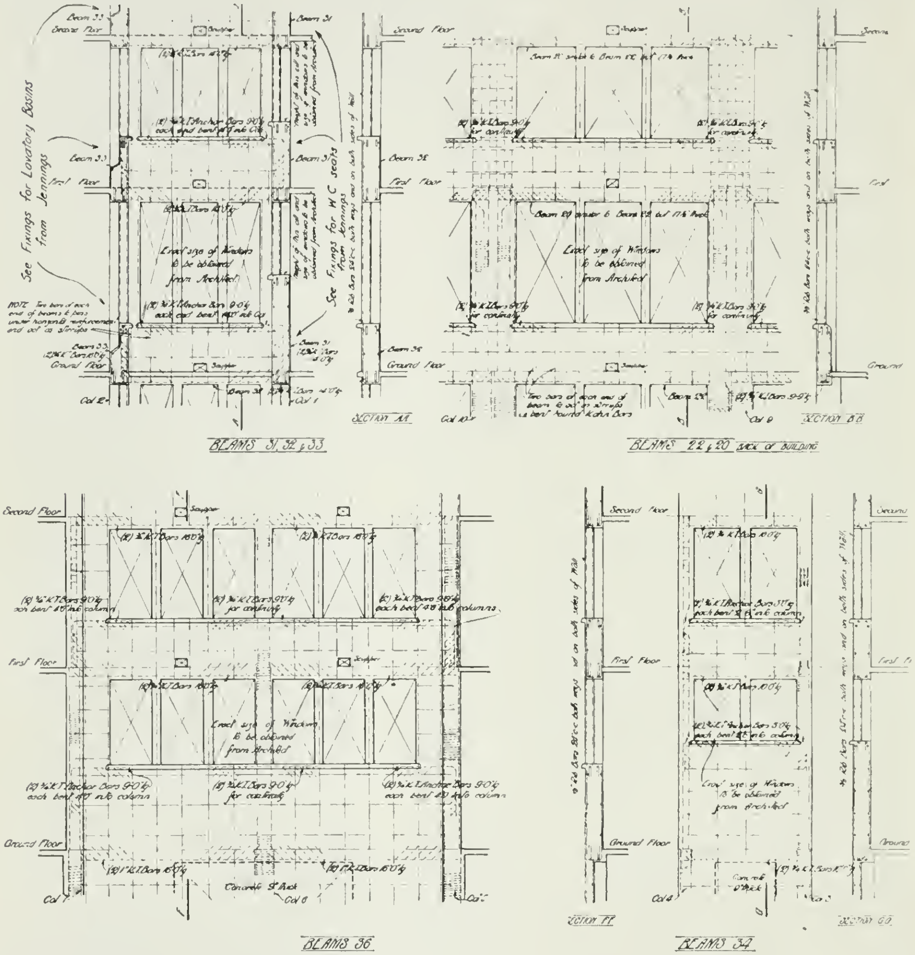


Fig. 4. Detail of Wall Beams and Piers.
Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.

the façade is invariably a covering quite distinct from the weight-carrying members behind.

The floors were designed to carry an external load of 3 cwt. per ft. super and were constructed with main and secondary beams and continuous slabs. The main beams were spaced at intervals of about 20 ft. and had a depth of about 30 in., while the secondary beams were spaced at 6 ft. intervals and had an average size of 26 in. by 12 in. The slabs had a thickness varying from

4 in. to 8 in., the general thickness being, however, 5 in. only, this including the granolithic finish, providing that it was as strong as the concrete employed for the slabs and that no cleavage plane occurred between the two. The reinforcement to the slabs consisted of ½-in. Rib bars at 9-in. centres, and these were put in in lengths of 26 ft. 6 in., thus extending through four bays of the floor.

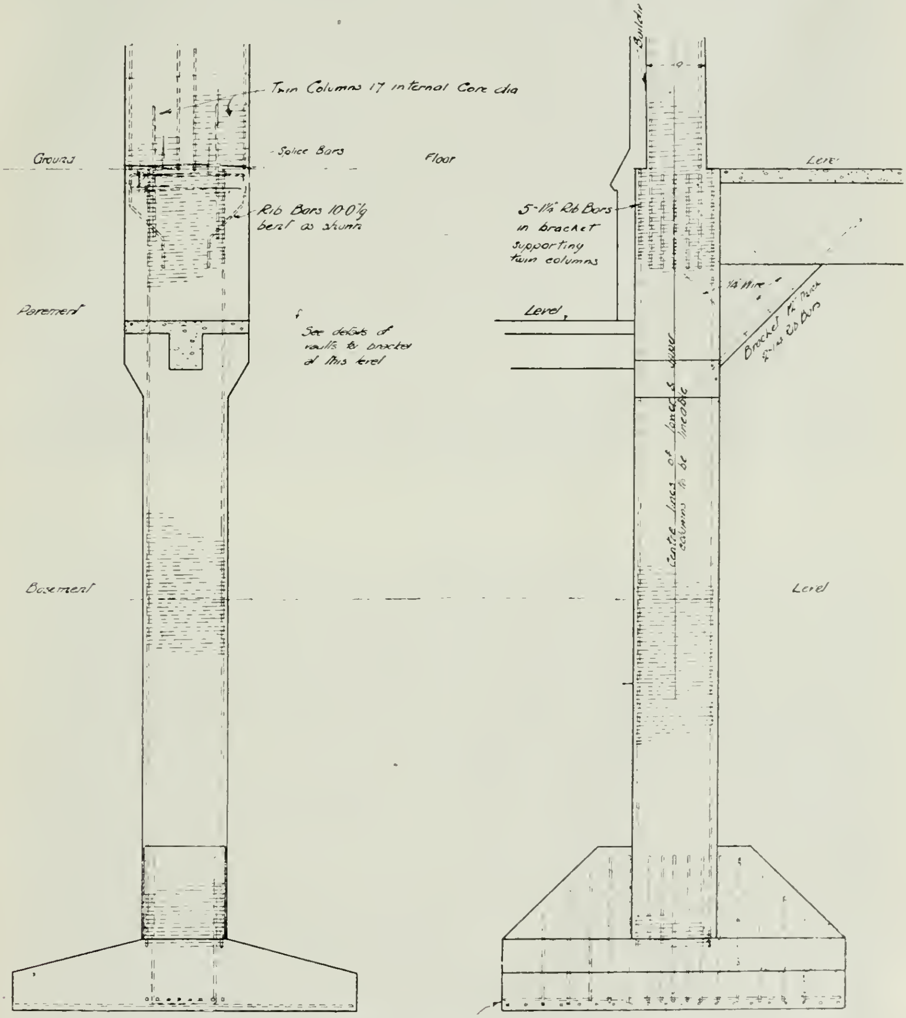

Twin Columns 17 internal Core dia
Splice Bars
Ground
Floor
Rib Bars 10·0% bent as shown
Pavement
See details of vaults to bracket at this level
Basement
Building
9
Level
5-1¼" Rib Bars in bracket supporting twin columns
¼" Wire
Bracket to have 5-1¼ Rib Bars
Level
Centre lines of lower & upper columns to be in line
Level

Fig. 5. Typical Elevation of Front Columns.
Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.

The slabs were cast in one operation with the beams, and continuity was ensured by ½-in. Kahn trussed bars 3 ft. 6 in. long, which were placed between the Rib bars over the secondary beams, this also giving provision against shear.

The sloping roofs are constructed with a thickness of 4½ in., and reinforcement is provided in the form of ⅜-in. Rib bars spaced at 12-in. centres. These slopes are carried by principals 9 in. thick spaced at about 10-ft. centres and reinforced with one ½-in. Rib bar and two ¾-in. Kahn bars. An example of a wall beam is that situated at the back of the publishing office, which is formed between the ground and first floor. This beam is 10 ft. deep and has a span of about 24 ft., with a thickness of about 10 in. The tensional reinforcement consists of two 1-in. Kahn bars and two ¾-in. Rib bars, the latter being turned up and carried to the top of the beam, ½-in. rib bar stirrups being also provided at varying distances apart hooked around the bottom reinforcement and wired at all cross points to prevent displacement while the concrete was deposited.

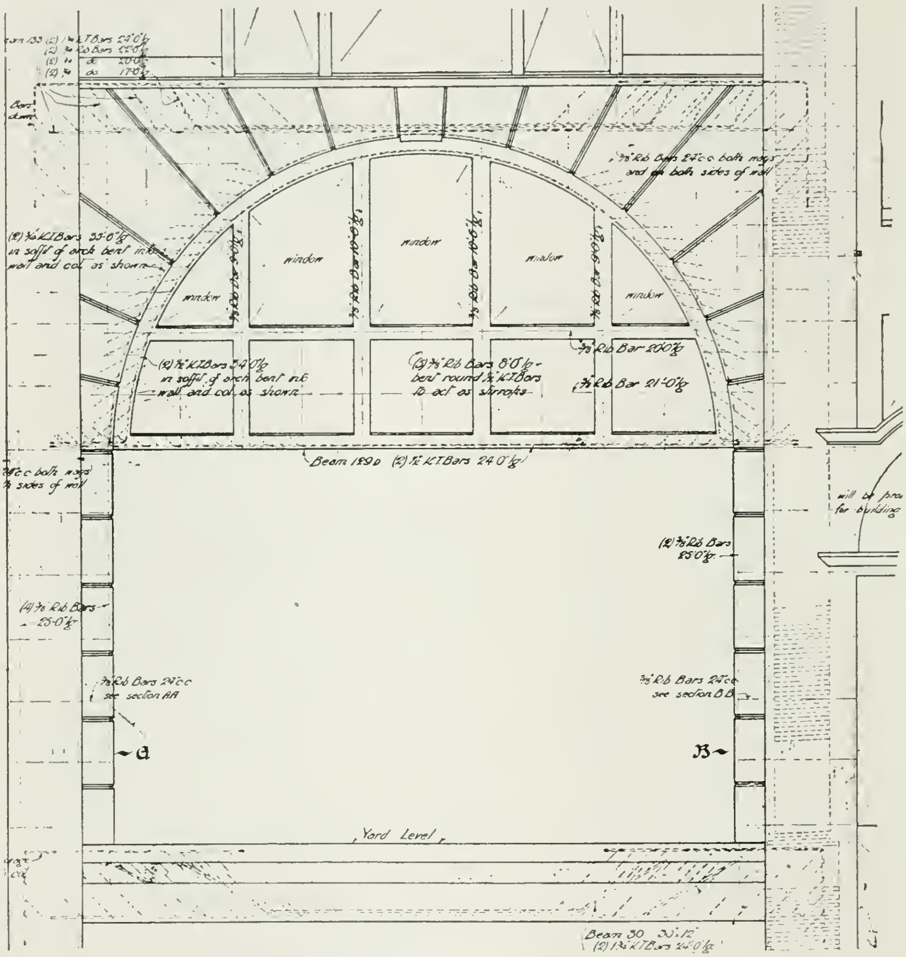

(2) ⅜ Rib Bars 22'0" lg
(2) ⅜ do 20'0" lg
(2) ⅜ do 17'0" lg
Bars down
(2) ¾ K.T. Bars 33'0" lg in soffit of arch bent into wall and col. as shown
window
window
window
window
window
⅜ Rib Bars 24" c.c. both ways and on both sides of wall
(2) ½ K.T. Bars 34'0" lg in soffit of arch bent into wall and col. as shown
(5) ⅜ Rib Bars 8'0" lg bent round ½ K.T. Bars to act as stirrups
⅜ Rib Bar 20'0" lg
⅜ Rib Bar 21'0" lg
Beam 129 D (2) ½ K.T. Bars 24'0" lg
24" c.c. both ways on sides of wall
will be pro... for building
(2) ⅜ Rib Bars 25'0" lg
(4) ⅜ Rib Bars 25'0" lg
⅜ Rib Bars 24" c.c. see section AA
⅜ Rib Bars 24" c.c. see section BB
A
B
Yard Level
Beam 30 30.12"
(2) 1⅛ K.T. Bars 24'0" lg

Fig. 6. Detail of large Openings.

Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.

The circular domes which appear on the front elevation have a diameter of

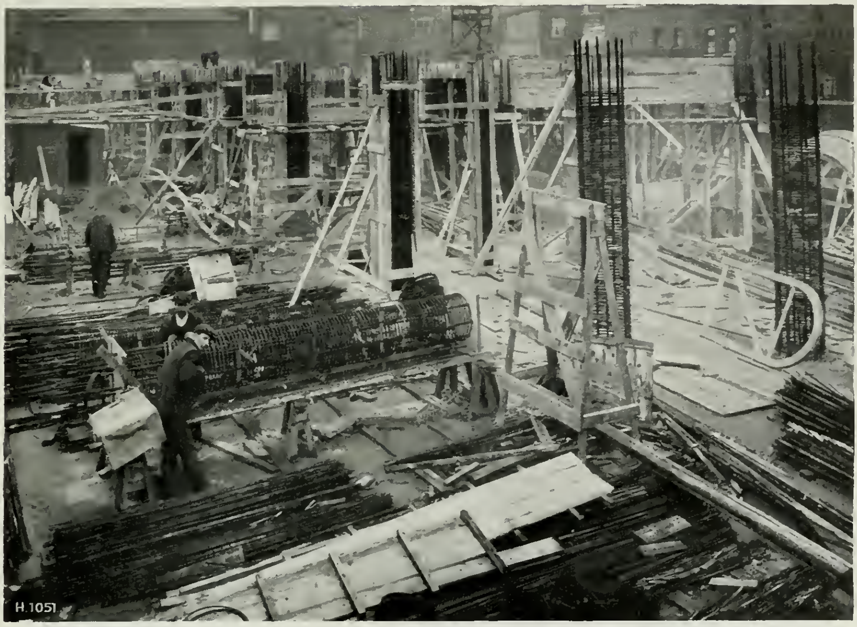


Fig. 7. View Showing Column Reinforcement.

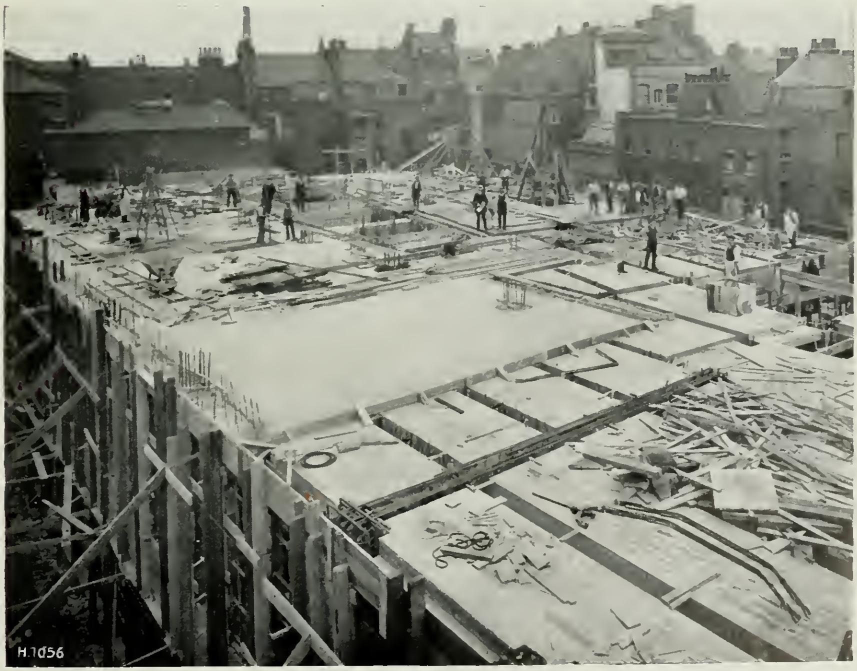


Fig. 8. General View of Work.
Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.

about 11 ft. and they are formed with 6 in. of concrete reinforced with circumferential and radial reinforcement of $\frac{3}{8}$-in Rib bars, while three $\frac{3}{8}$-in. Rib bars are placed at the springing to act as a tie, these having a lap of 12 in.

The staircases are formed with landings 4 in. and 6 in. thick, reinforced with Kahn bars, while the flights are constructed as slabs $4\frac{1}{2}$ in. and 6 in. thick, the reinforcement being carried well into the side walls, which are also formed in reinforced concrete.

The concrete throughout was composed of 4 parts of clean ballast, gauged to pass a $\frac{3}{4}$-in. mesh, 2 parts of sand, and 1 part of Portland cement. The building must be considered as an excellent example of a complete concrete building, and the material has been employed in a consistent manner, which exhibits its adaptability to any form of construction when designed in an efficient manner.

The contractors for the work were Messrs. D. G. Somerville & Co.

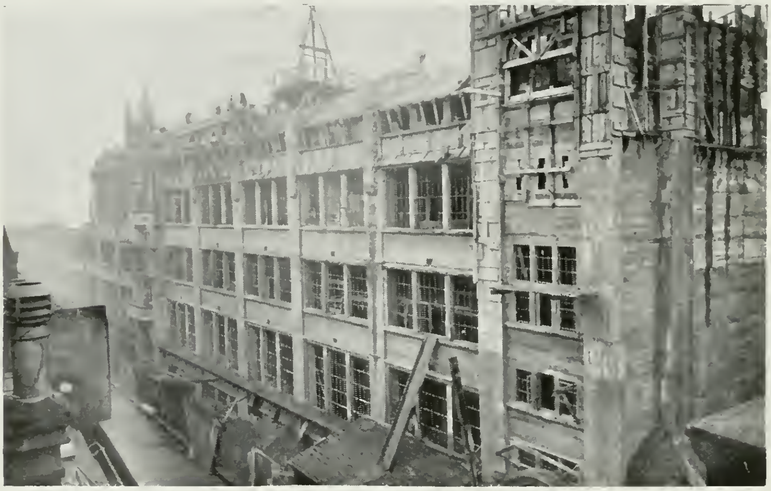
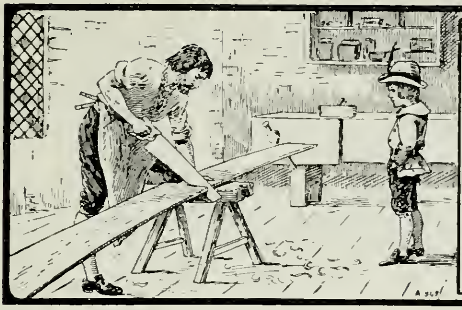


Fig. 9. View in Course of Construction.
Reinforced Concrete Building for Messrs. R. Clay & Sons, Ltd.



FLOOR AND COLUMN FORMS for a TYPICAL ALL REINFORCED CONCRETE BUILDING

By H. C. JOHNSON,
Demonstrator in Engineering, University College, Cork.

We are giving a further article on the subject of "Forms for Reinforced Concrete Work," as this is a subject worthy of special attention, owing to the saving which can be effected if the formwork is economically and efficiently carried out.—ED.

THE reasons for the difference in bids on a given piece of reinforced-concrete work are three in number :—

Firstly and mainly.—The difference in the making, placing and removal of formwork.

Secondly.—The failure to make a proper study of form design, the old rule-of-thumb methods being applied to a new form of construction, often without any previous experience of such work.

Thirdly.—The failure so to arrange the work that good and experienced men are kept going, instead of being put off for a week or two after each storey of formwork is completed, with the result that some new men, with no previous experience, have to be taken on to help work the next floor.

It is the aim of the writer to present a system of formwork which is a combination of two systems as used by two American reinforced concrete companies, firms that keep costs for each portion of the work to estimate from, and to make one job check another. Their men have to work methodically and quickly, for which they are paid accordingly, since they are specialists.

The writer helped in no small way to develop these types of formwork, but does not consider that the type to be presented is new in its main principles, although the details may be.

Since the total cost of the formwork for a building varies from 25 per cent. to 50 per cent., or even more, of the total cost of the completed structure, and a saving of 5 per cent. on the formwork means 5 per cent. more profit, it is easily seen where the saving in money can be made; in fact, it is the first item to look to, for the cost of providing and placing a cubic yard of concrete or a ton of steel does not vary anything like so much as similar work on forms.

Naturally, the more regular the building and the more nearly all storeys are alike the cheaper the formwork, and therefore the finished structure.

Therefore in designing the structural parts of a reinforced-concrete building the aim should be to keep the size of all beams and girders the same and only change the column sizes each second and even third storey. A little more concrete is cheaper than the change of formwork which would have to be made in the column, girders, beams and slab, for it costs as much to change the column two inches as six.

Here it would be as well to point out that *girders and beams* is a more simple way of speaking and working by than *main beams and secondary beams*, as is also the method of lettering and numbering shown in *Fig.* 1; this applies as forcibly to the members themselves as to their forms, which should be numbered to correspond, so that

"*B*" is used for members supporting slab,

"G" is used for members supporting beams,

while the change in number is made when the size of the member or the steel (length or size) is different.

Where a firm does considerable reinforced concrete work a "standard" for formwork should be made and copies handed to the draftsmen, the general and carpenter foremen, and also to the carpenters, if they are kept on steadily, and a few one-hour lectures given, when they could be explained to the men.

The principal difficulty about setting such "standards" is that where a contracting firm do not do their own designing they find, for instance, that where one specialist runs his beams and girders into the columns square, another must have knee braces or brackets, so that the standard form system is often condemned. This would indicate that those firms doing their own designing would have the best chance of obtaining the contracts, unless the specialist is also willing to adopt some standard.

With those working in reinforced concrete there should be perfect co-operation. The designer of the structure in the office should know the practical as well as the theoretical side of construction; should make his beam and girder widths and depths to suit the commercial sizes of timber, and so avoid unnecessary ripping of the material; keep girders and beams the same size in succeeding storeys, unless very different live loads are specified, when the depth could be easier increased than the width.

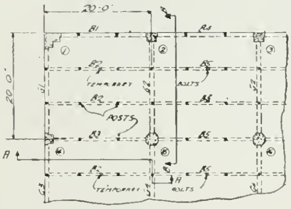


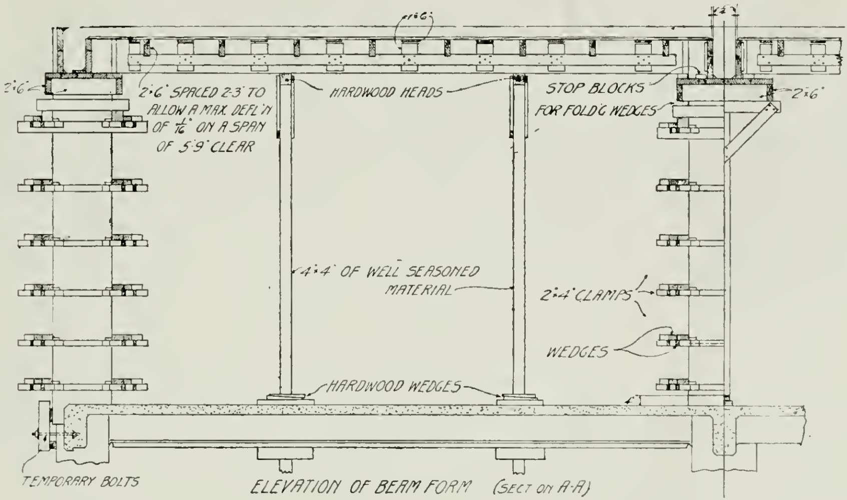
FIG. 1

Fig 1.
General Lay-out of a Typical Bay of Three Panels.

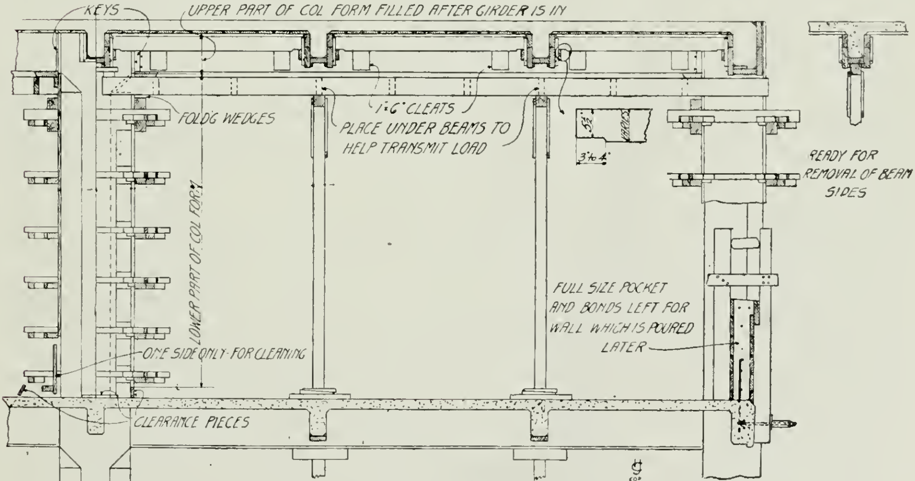
It does not matter whether a building is regular or irregular, a "standard" would apply to both, for more saving is effected by a uniform system than by varying the methods, because good men get into the swing of a set system and naturally accomplish more, and therefore waste less material.

The greatest mistake is made by some contractors in thinking that by setting labourers to remove the forms they are saving the difference between their pay and that of carpenters for the same period. The labourers' main idea is not systematic removal of the forms, but quick uncovering of the concrete, "to let it dry so that it can harden." Like a lot of other people, they imagine that keeping concrete wet prevents it becoming its hardest, whereas the reverse is the case. The result of such energy is that cleats are split, beam sides are in two or three pieces, nails left sticking out at awkward angles, and the whole of the formwork lying on the floor in an indescribable state, to be left there, perhaps, overnight to warp out of shape.

This damage has all to be made good by the carpenters in the next storey. Moreover, new material will be required for patching, whereas the forms should have gone at least four storeys before patching was necessary. Then, again, formwork is often made too light, the idea being that it is easier handled, and saves timber, but the maximum spacing of posts, etc., is limited by the thickness of boarding they are carrying; light work increases the


1½·6
1½
2·6
2·6 SPACED 2-3 TO ALLOW A MAX. DEFL'N OF 1/16" ON A SPAN OF 5'9" CLEAR
HARDWOOD HEADS
STOP BLOCKS FOR FOLD'G WEDGES
2×6
4·4 OF WELL SEASONED MATERIAL
2·4 CLAMPS
WEDGES
HARDWOOD WEDGES
TEMPORARY BOLTS

ELEVATION OF BEAM FORM (SECT ON A-A)
FIG. 2


KEYS
UPPER PART OF COL FORM FILLED AFTER GIRDER IS IN
FOLD'G WEDGES
1·6 CLEATS
PLACE UNDER BEAMS TO HELP TRANSMIT LOAD
5½
VARIES
3" to 4"
READY FOR REMOVAL OF BEAM SIDES
LOWER PART OF COL FORM
ONE SIDE ONLY - FOR CLEANING
FULL SIZE POCKET AND BONDS LEFT FOR WALL WHICH IS POURED LATER
CLEARANCE PIECES

ELEVATION OF GIRDER FORM (SECT ON B-B)
FIG. 3

chance of sagging of beams or slabs or bulging of columns, resulting in concreting being stopped while the forms are wedged up into line.

Fig. 1 shows the general lay-out of a typical bay of three panels, for which this formwork is designed, the slab thickness being 4 in., beams 6 in. by 10 in. and girders 8 in. by 14 in. under the slab.

In *Figs.* 2 and 3 it will be noticed that the posts are wedged at the bottom, and provided with hardwood wedges and heads—the wedges should always be so placed, for this obviates the necessity of using trestles to adjust the beam for line, as would be the case if they were at the top. The wedges under the girder "bridge" are easily got at by using the column clamps as a ladder. The hardwood head and wedges increase the carrying value of the posting, and, what is more important, the load is applied axially to the post and no bending is induced as would occur when the crosshead was bolted to the *side*. This difference in carrying value is best illustrated by tests made recently in the Laboratory of University College, Cork. A 7 in. by 3 in. spruce post, with a 7 in. by 3 in. hardwood head bolted on by two $\frac{5}{8}$-in. bolts, as shown by *Fig.* 4, stood only 3·125 tons (which for 21 sq. in. equals 333 lb. per sq. in.), while a 4-in. by 4-in. spruce post, with a 4-in. by 4-in. hardwood head, as used in this formwork, stood 18·3 tons (which for only 16 sq. in. equals 2,560 lb. per sq. in.).

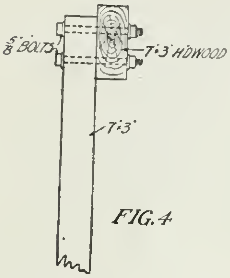


FIG. 4

Bolted Head (poor method).

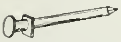


FIG. 5

Special Nail.

The posts were 10 ft. high over all and loaded until the deflection was about 6 in. In the case of the 7 in. by 3 in. the deflection started at 2 tons and in the 4 in. by 4 in. at nearly the full load.

From the above it can be seen that by putting the weight axially on the post it will carry at least five times as much as if placed eccentrically, and therefore fewer posts are needed, providing the members being supported will span the increased distance safely. The hardwood head and wedges are very necessary, because spruce would have been crushed at about 5 tons, since it would have been used *across* the grain, a very different thing from *along the grain*.

The girder bridge made strong enough to support the weight of three or four men and to span from column to column is a distinct advantage, for as soon as the column forms are erected these "bridges" are raised and correctly placed, a man can then go out and lower a rope, which is made fast to the end of the beam form, which is then raised and lightly tacked, the nail being so placed that it is easily accessible and with the head outstanding a $\frac{1}{4}$ in. This applies to all nails used, except where permanent cleats occur.

Special nails for formwork are made and are as shown by *Fig.* 5.

The under joists in these bridges are made long enough to run within 3 in. of the centre of the column and placed far enough apart to clear the formwork for a 26-in. by 26-in. column. They rest (through the 4-in. by 4-in. cross-pieces and the wedges) directly on the column clamps, all of which act as cleats for the column boarding, which is permanently nailed to them, the top set being 4-in. by 4-in. material to allow them to be securely fastened to the boarding and so carry the loads transmitted to them by the girder and beam forms, which are adjusted by means of the wedges under the two cross-pieces. This arrangement ensures a constant relation between the beams and girders, as does also the girder bridge at the other beams, since the blocking pieces at the column and on the bridge remain unchanged so long as the beams and girders remain the same.

The column clamps on the girder side are placed so as to cross under those on the beam side. This allows the girder side of the column form to be dropped and removed as soon as the wedges have been struck and the clearance pieces at the floor level removed. These girder sides run only to the under side of the girder proper in one piece, that portion above is filled in separately with cleated material. *Fig.* 7 shows this arrangement.

The beam sides of the column form run to the under side of the slab sheeting, except where cut away to allow the beam connection and a clearance on each side sufficient to allow it to pass the cleats on the side of the beam form. This space is filled, as shown in *Fig.* 7, by a loose piece and a key.

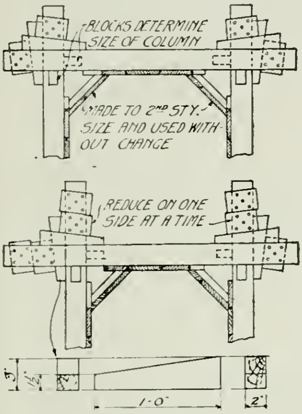
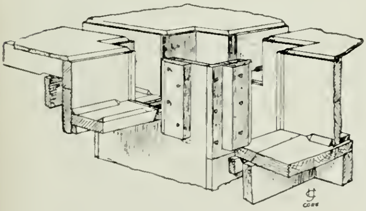

BLOCKS DETERMINE SIZE OF COLUMN
MADE TO 2ND STY. SIZE AND USED WITHOUT CHANGE
REDUCE ON ONE SIDE AT A TIME
3"
1½"
1-0"
2"

FIG. 6

Column Form Before and After Reducing.

Such a column form is easily removed without disturbing the posts under the beams and girders, and without any risk, since if the column can stand without forms, so can the small length of beam and girder which are still supported by the posts at the vital points.

The reason for early removal of the column forms is that they may be cleaned, altered to suit the next storey size, patched if necessary, oiled or soaped, then hoisted to the next floor and erected and so keep the carpenters going—not with the idea of allowing the concrete to dry.



HEAD OF COLUMN FORM

FIG. 7

Column forms should never be removed under three (whole) days, and only then in the lower storeys, where, of course, the columns are larger and capable of carrying all the load one floor could put upon them, since they are designed to carry floors above, whereas the beams and girders support only one floor. Therefore the nearer the top of the building the longer they should remain in position.

It is often dangerous to give only a minimum time before removal of forms, because that minimum is gradually applied to all the formwork, whether the weather be warm or cold. Temperature has the greatest effect on the setting time of concrete and moisture the greatest effect on strength (except the quantity and quality of cement), therefore three days when the temperature is 60° to 70° has just about the same effect as five days at 35° to 45°.

The removal of the girder, beam and slab forms can usually be made as these are required for the next storey, providing the beams and girders are supported as in the lower parts (storey below) of *Figs.* 2 and 3, where the original beam bottoms are left in as long as posting is required.

It will be noticed that the column clamps are closer together at the bottom. This is very necessary, for the pressure there while the concrete is being poured is probably twice that of water pressure for the same head. This is a point much disputed, but the writer intends, at an early date, to make some tests regarding such pressures on formwork; but the first clamp should be about 6 in. (to the under side) above the floor line, and for a 12-ft. head, as in this case, using $\frac{7}{8}$ in. sheeting (nominally 1 in.) the spacing should vary from 15 in. at the bottom to 24 in. at the top; if heavier sheeting is used in order to reduce the number of clamps, then the clamps must be made much stronger than those shown—not that these are very light, since they are made to suit the sheeting thickness, for it would be folly to put in strong clamps widely spaced with thin sheeting.

The changes in column size should be made by removing the sheeting that would prevent the clamps being brought up tight again, and which should have been so placed in the first making that no ripping is necessary. This, naturally, means setting the columns out to scale on paper and planning the boarding to suit, a matter of an hour at the most *before starting* the job, but a saving of many hours *after the start*.

While on the subject of column forms, it would be as well to describe how the first and outside ones are held in position. In the first storey they would be stayed by braces fixed to pegs driven into the ground, but some other arrangement is necessary when starting off a concrete floor. A $\frac{1}{2}$-in. bar is threaded both ends for a distance of 3 in., one end is provided with a nut and washer; this end is then placed in the concrete in the middle of the beams shown on *Fig.* 1 while concreting, with about 6 in. outstanding. By placing the bolts in these particular beams the posting is avoided by the braces. Just before the column forms are to be erected, a piece of 4 in. by 4 in. with a $\frac{1}{2}$ in. hole is placed in position and held by a nut and washer screwed on the outstanding bolt and the braces spiked to these. When the block is no longer in use it is removed, the bolt unscrewed, leaving the nut and washer embedded in the concrete, and the hole filled with grout. A similar arrangement is adopted to hold that portion of the pilaster form which runs below the floor line as shown in *Figs.* 2 and 3.

If spirals are used for column reinforcement then the columns should be made octagonal in cross-section to save floor space. This means very little additional work; in fact, it is fully compensated for by the saving in concrete, thus proving how small an item it is as regards cost, while the appearance of the interior is much improved.

It will be noticed that 2 in. (actually $1\frac{3}{4}$ in.) material is used for beam sides and bottom and girder bridge and sides. This will prove economical even if the formwork is only used twice and then returned to the yard, for while it is easy to take any warp out of the slab sheeting and column forms by reason of the weight of concrete in the former and the pressure of concrete and the clamps in the latter, it is seldom possible to straighten a beam side.

The main reason for 2 in. material is that the beam sides have to act themselves as beams, since the slabs as well as the beam itself must be supported by them. For this reason they should be cleated with good broad material about 6 in. to 8 in. wide and at least ⅞ in. thick and nailed in a systematic manner with clinched nails.

The girder bridge should also be made up strongly, for, like the beam sides, it is a piece of formwork that is never altered.

The strips supporting the beam bottom are adjustable in case of an increase or decrease in depth, and should therefore be screwed on.

The fillets forming the splay on the beams should be nailed to the beam bottom, not to the sides, or the beam sides would not come away easily; those for the girders should be nailed to the sides if possible.

The ledgers on the outside of the beam sides should be permanently nailed to the cleats with their tops exactly 5½ in. below the under side of the slab sheeting. This allows the joists supporting the sheeting to be "sized" as indicated in *Fig.* 3, for 6 in. is only the nominal size of the timber. By using these ledgers the fiddling little blocks often used to support the joists, and which have to be adjusted to suit a different joist at each move, are done away with and the joists can be so placed that they avoid the sheeting cleats, or closer together if a heavier slab has to be supported, when, of course, the girder bridge and posts may have to be altered to carry the extra load.

The slab sheeting is the portion of the formwork that suffers most during removal, the reason being that too many and too large nails are used to tack it to the joists or to the beam and girder sides. It is only experienced men who know how to do this tacking correctly, nevertheless it is only necessary to use 1½-in. brads, oval preferred, for ⅞-in. sheeting, or 2 in. for 1¼-in. sheeting, and if a contractor is wise he would only allow this size on the upper work; larger nails are, of course, necessary for braces, etc., down below. This tacking should only be done where actually needed, avoiding, if possible, the beam and girder edges, because here the nails would have to go through the splay (which should be run all round, setting back ¼ in. as shown) and would be driven into the beam sides at least 1 in., with the result that when the beam and girder sides were pulled splinters would be broken off, due to the nail heads pulling through the timber, while if put into the joists they would remain in the sheeting when the joists were removed, to be easily driven back and pulled.

The slab sheeting should be of tongued and grooved material, as for the column forms, for it is often exposed to the sun over a week-end, when considerable shrinkage would take place, which, if on plain boards, would either call for much extra work or else a considerable loss of cement from the concrete, which would spoil the surface of the floor below, besides decreasing the strength of the slab and the protection of the steel reinforcement.

The object and advantage of the keys are easily seen when one considers how the beam and girder sides are freed for removal by their use, but it is necessary to point out that the nails must be left outstanding ready for easy pulling and that the men should be instructed to tack them lightly to the beam sides to which they belong, so that they be ready at hand when needed for the next storey, instead of throwing them to the floor to be used for firewood—as a matter of fact, any man found burning or cutting up any keys, wedges, clearance pieces, or sole plates should be discharged, for there is a vast difference between doing this and using joist ends left from ordinary construction work, since burning is their ultimate end in any case.

Since the changes in column size must take place patching is necessary in the beam, girder and slab forming. This is a simple though tedious process, but the keys should be retained, otherwise the material used to extend the beam and girder sides and bottoms, which must necessarily be put with the grain vertically, would bind the formwork because of swelling due to absorption of water.

A very important part of the work is the method of removal, and this should always be carried out in a careful manner, for more collapses are due to thoughtless work on this item than anything else—except, perhaps, bad formwork.

I am not prepared to add anything to what has already been said regarding time of removal, for it depends on so many things, the chief of which are:

1. The cement—for I know of two cements, one of which was as strong at 28 days when mixed with three parts "standard sand" as the other was neat.
2. Temperature.
3. Weight and position of members.
4. Whether hand- or machine-mixed, for the hand-mixing speed varies so much that some partly set material may have been used.
5. Long or short span—that is, in proporation to depth. A short span would be able to carry itself sooner than a long one, because it would, no doubt, have been designed for a heavier live load.

There is no doubt that it is dangerous to give definite removal times without knowing the job and conditions, so even if this article appear incomplete the writer is not tempted to go further than the three days minimum in summer or five in winter for column forms in this climate.

ORDER OF REMOVAL.

1. Wedges in column forms struck and placed in sacks for hoisting to next floor.
2. Clearance pieces.
3. Sides of column form under girder.
4. Keys at beam sides.
5. Sides of column form under beams.
6. Post up beams as close to girder as possible with new posts.
7. Remove posts under girder bridge.
8. Remove bridges.
9. Place new posts under girders, near beams, with cross-heads running *along* girder bottoms, in same position as original posts.
10. Prop up slab joists temporarily as near beams as possible.
11. Draw nails holding beam sides to beam bottom.

12. Loosen wedges under posts supporting beams and turn cross-heads lengthwise and wedge up again.

13. Remove beam sides.

14. Remove girder sides.

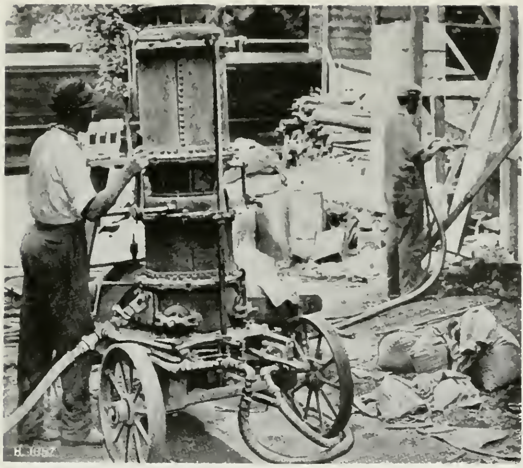
15. Remove slab sheeting and joists.

These operations, as pointed out before, would extend over a number of days and the material should only be pulled as required for the next storey, otherwise the work is made more confusing. The whole of the formwork should be marked.

Do not allow steel to accumulate at one place on the forms, otherwise they will be overloaded.

Green timber is best where it comes in contact with the concrete, but seasoned material will deflect less and should be used for posts, clamps, etc.

This article covers only a small portion of concrete formwork, but that portion which is, perhaps, most often used, but which does not get as much study as it merits.



THE CEMENT GUN.

A METHOD OF APPLYING CEMENT MORTAR BY COMPRESSED AIR.

By WILLIAM A. JORDAN.

A paper on the subject of "The Cement Gun" was read by Mr. William A. Jordan before the American Society of Engineering Contractors, from which the following particulars of this novel method of applying cement mortar are taken. The photographs from which our illustrations were made were placed at our disposal by the General Cement Products Co. of New York.—ED.

The methods of placing concrete in construction are, relatively, not as far advanced as the excellency of the material itself warrants. Cement as at present manufactured is a reliable and uniform material possessing definite and well known characteristics. To advance the position that concrete occupies among the building materials of modern civilisation requires only the discovery and exploitation of some method of application that will ensure in the finished work the uniformity and strength known to be possible, but in practice never realised.

Among the various materials used in building construction, hydraulic cement appears to be one for which machine application might be most rationally expected, and, accordingly, it is with this material that inventors have experimented more extensively than with any other.

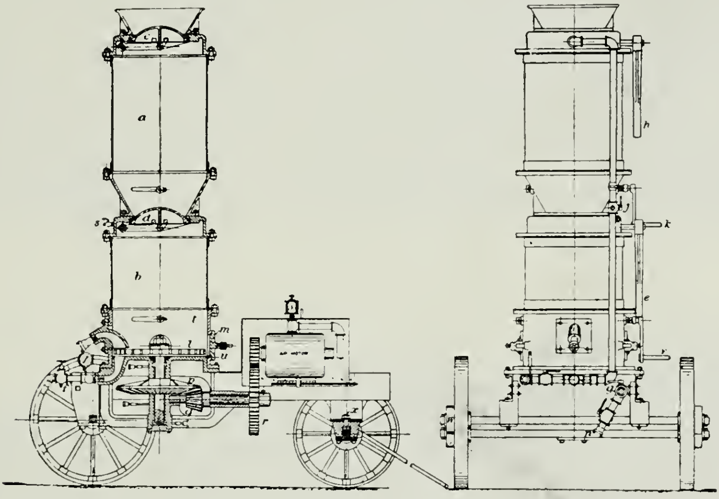
The solution of the problem came with the discovery of the so-called "cement gun." As a matter of fact, this is not a "gun" in the ordinary meaning of the word, but it was so designated from the fact that the material is projected from the nozzle with considerable force and literally "shot" into place.

The cement gun consists essentially of a hopper into which the cementitious materials are placed, a hose connected to the bottom of the hopper through which the dry mixture is forced by compressed air, and a specially constructed nozzle at the end of the hose to which another hose supplying water is attached for hydrating the material. A diagram of the gun is shown on page 343, and its working will be explained in detail later.

In the cement gun process, the dry materials are forced through the hose by means of compressed air, hydrated at the nozzle, and shot into place with a muzzle velocity corresponding to a pressure of 35 lb. per sq. in. The resulting product is of a quality never before attained by hand or any other machine process. The combinations of the elements, cement and water, necessary to produce the plastic material, takes place in transit, and the latter is not disturbed after it is placed.

It is a well known chemical fact that the instant water is brought into contact with cement, the initial set, or chemical combination, begins. The initial set must be looked upon as the commencement of that hardening which gives the mass its ultimate strength. Hence, subsequent manipulation or handling tends to disturb this initial set, and, consequently, must weaken the product. These objectionable features are entirely overcome by the cement gun process; the hydration taking place in transit and immediately before and during emplacement, and the chemical combination or initial set taking place where it belongs—*i.e.*, in its final resting-place, and not on the mixing board.

The cement gun employs only the necessary amount of water for the hydration of the cement, and because the materials are projected with considerable force, all surplus water and air are expelled. The product is a non-porous, impermeable mass, possessing the maximum density, which, by reason of these qualities, is practically waterproof.



Cross Section of Cement Gun.

When considering "gun" made concrete, it should be borne in mind that the powerful air blast imparts a great velocity to the particles of sand. By means of the sand blast, ordinary dry sand will wear its way through iron or steel with incredible speed. In the cement gun the high velocity imparted to the mixture of sand, cement, and water as it is thrown upon a surface to be coated is of the utmost importance in producing a concrete of maximum density, as will be explained further on.

It was shown that to secure the strongest concrete a given cement is capable of producing, water sufficient only for the actual hydration of the cement contents should be added, because any surplus water which may remain weakens the mass by creating voids.

In the cement gun, as explained previously, the sand and cement are first mixed together in correct proportion while dry. This dry compound is deposited in the hopper, from which it is fed into a powerful blast of air which conducts it through a suitable length of ordinary rubber hose until the nozzle is reached.

The Sea Wall at Lynn, Mass., showing Destructive Work of Tides.

The Sea Wall at Lynn, Mass., Repaired by the Cement Gun.

Here, in the form of a fine spray, the water necessary for hydration is added at a pressure slightly in excess of the air pressure. The velocity with which the materials are travelling, however, is so great—300 ft. per second—that hydration in reality takes place in the air, after leaving the nozzle, and is completed as the mixture is deposited. It is, of course, impossible to regulate the water thus admitted to precisely the amount theoretically required, as a very small variation in the air pressure alters the amount of dry material flowing from the machine; nor is such fine adjustment necessary or advisable. The mixture of sand, cement, and water strikes the object or surface on which it is deposited with such force that any surplus water contained is continually driven to the surface and flows away.

The cement gun has been used on a number of engineering projects very successfully. Probably the most prominent place where the gun has been used is the Panama Canal. Here it was employed to coat the disintegrating rock of the Culebra cut with concrete. The excavation through the Culebra cut at this point uncovers the disintegrated clay. As the underlying clay disintegrates, it allows enormous masses of the rock to fall down into the excavation, and the deeper it goes the more rock caves in. The problem was to hold the rock back. The engineers' original thought was to build a flat wall through the cut. Their idea was to excavate and reinforce with old French railroad iron. This revetment wall varied in thickness from 4 in. to 4 ft., depending on how much excavation had to be done. A cement gun mounted on a flat car, with five men in the crew, squirting concrete from the nozzle, followed the contour of the rocks, coating it to a thickness of about 2 in.; it was not necessary to build up the concrete 4 ft. thick, or to have a plain, smooth surface, as given by this wall.

The New York Central and Hudson River R. R. Co. are using the cement gun process for coating the steel work in their new 42nd Street terminal. This work has been done without forms, or centering of any kind, and a coating of two to three inches of concrete has been applied to all structural members.

The Dover Street Bridge in Boston was repaired by means of the gun process. Here it was necessary to contend with tides, and it was found that concrete placed with the gun was unaffected by the tides after it had been in place three or four hours. In experimental work a hose has been turned upon cement gun work after being allowed to stand 2½ hours, and it was unaffected.

The sea wall at Lynn, Mass., was repaired by Mr. J. R. Rablin, Chief Engineer of the Metropolitan Park Commission, in a very satisfactory manner by means of the cement gun. This work is shown on page 344. Here also it was necessary to contend with the tides, but it was found that they had no effect when the cement was once in place.

The gun holds about 3 cubic ft., its capacity is about 10½ to 12 tons of material in eight hours, and the materials are mixed by hand or with a machine before they go into the gun. In a large operation a mortar mixer was used. It is necessary to thoroughly mix and screen the material, otherwise foreign matter gets into the feed wheel and chokes it up. The men who put in the material are ordinary labourers, but the man at the nozzle must be experienced.

FORMULAE FOR THE VERIFICATION OF SCHEMES IN REINFORCED CONCRETE.

By MAURICE BÉHAR, C.E.
(*Civil Engineer of the Ecole Nationale des Ponts et Chaussées*).

THE method of verification given in the October number of CONCRETE AND CONSTRUCTIONAL ENGINEERING was somewhat lengthy. It is the author's intention to set forth, in the following article, a method which has the advantage of being much quicker than the one previously described.

This method is based upon the theory of the calculation of bodies subjected to bending, and was originally established by Mr. N. de Tedesco, civil engineer, of Paris.

I do not think it necessary to expose this theory in the present article, as the reader will find same completely developed in the "Manuel Theorique et Pratique du Constructeur en Ciment Armé," by Messrs. N. de Tedesco and V. Forestier. (Editeurs, Beranger, Paris.)

I shall simply give the fundamental formulae established by the above-mentioned authors, and it is my intention to show how these formulae may be applied to the verification of a scheme in reinforced concrete.

FUNDAMENTAL FORMULAE.

Floor Slabs.—The position of the neutral axis is given by the formula

$$y = \frac{mcd}{t + mc}$$

If we assume that the extreme compressed fibre of the floor slab is stressed at the rate of 600 lb. per sq. in., and the steel in tension at the rate of 16,000 lb. per sq. in., we obtain, by substituting the letters by their values in the preceding formula,

$$y = \frac{15 \times 600}{16000 + 15 \times 600} \times d$$

Hence (1) $y = 0{\cdot}36d$

The lever arm of the bending couple

$$z = d - \frac{y}{3} = d - \frac{0{\cdot}36d}{3}$$

Hence (2) $z = 0{\cdot}88d$

The formulae (1) and (2) will allow us to verify any floor slab.

Beams.—The position of the neutral axis will be given, as in the case of the floor slab, by formula No. (1). The lever arm of the bending couple is given by the formula

$$(3) \quad z = d\left(1 - \frac{n}{3} \times \frac{1{\cdot}08 - 2n}{0{\cdot}72 - n}\right)$$

In this formula d is the theoretical height of the beam, n the ratio, $\frac{e}{d}$ between the thickness of the slab and the theoretical height of the beam.

The application of the formula (3) in each particular case is rather lengthy.

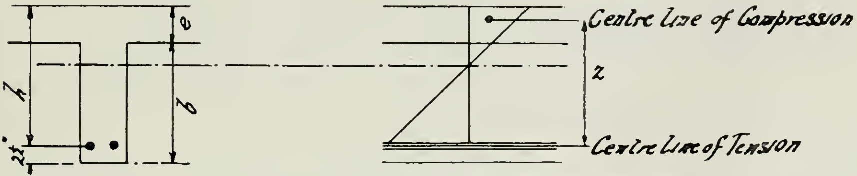
In order to simplify the application of this formula, I give herewith a table giving the various values of z measured in feet, and for all beams having a height varying between 6 in. and 36 in. measured from the soffit of the slab.

This table is applicable to beams having a slab of a thickness varying between 3 in. and 6 in.

Concerning beams not included in this table, the verifier should have recourse to formula No. (3).

Table Giving Lever Arms "Z" of Beams.

$$h - b + e - 2\tfrac{1}{2}'$$



Height of Beams.	Thickness of Slabs.						
	3 in.	3½ in.	4 in.	4½ in	5 in.	5½ in.	6 in.
in.							
6	0·474	0·510	0·547	0·583	0·620	0·656	0·693
7	0·547	0·583	0·620	0·656	0·693	0·729	0·766
8	0·620	0·656	0·693	0·729	0·766	0·802	0·838
9	0·697	0·730	0·766	0·802	0·838	0·875	0·911
10	0·776	0·807	0·840	0·875	0·911	0·948	0·984
11	0·856	0·885	0·917	0·949	0·984	1·021	1·057
12	0·936	0·964	0·995	1·026	1·059	1·094	1·130
13	1·018	1·045	1·074	1·104	1·136	1·169	1·203
14	1·098	1·126	1·154	1·184	1·213	1·246	1·278
15	1·183	1·207	1·235	1·263	1·293	1·323	1·355
16	1·265	1·289	1·316	1·344	1·372	1·403	1·432
17	1·346	1·371	1·397	1·425	1·453	1·481	1·512
18	1·429	1·454	1·479	1·506	1·534	1·562	1·591
19	1·512	1·536	1·561	1·588	1·615	1·643	1·671
20	1·593	1·617	1·644	1·669	1·696	1·724	1·751
21	1·675	1·700	1·726	1·751	1·778	1·804	1·832
22	1·758	1·783	1·807	1·834	1·859	1·885	1·913
23	1·841	1·865	1·890	1·917	1·941	1·968	1·994
24	1·925	1·948	1·973	1·998	2·024	2·049	2·075
25	2·006	2·030	2·055	2·080	2·107	2·131	2·158
26	2·091	2·113	2·138	2·162	2·188	2·214	2·239
27	2·173	2·196	2·220	2·245	2·270	2·297	2·321
28	2·256	2·280	2·308	2·327	2·352	2·379	2·404
29	2·339	2·363	2·385	2·410	2·435	2·460	2·487
30	2·422	2·446	2·469	2·493	2·517	2·542	2·569
31	2·505	2·529	2·552	2·575	2·600	2·625	2·650
32	2·589	2·611	2·636	2·658	2·682	2·707	2·732
33	2·672	2·694	2·719	2·741	2·765	2·790	2·814
34	2·756	2·777	2·801	2·824	2·848	2·872	2·897
35	2·839	2·860	2·884	2·908	2·930	2·955	2·979
36	2·923	2·944	2·967	2·991	3·013	3·037	3·062

Numerical Applications.—Let us take, as an example of flooring, the same floor as has already been given in my first article of October last.

Floor Slab.—We have found for the slab a maximum bending moment of 1,038 ft.-lb. To counteract this moment a thickness of $4\frac{1}{2}$ in. was provided, and a section of steel in tension of 0·286 sq. in. per foot run of slab.

The theoretical height *d* is therefore

$$d = 4\tfrac{1}{2}'' - 1'' = 3\tfrac{1}{2}''$$

The position of the neutral axis is therefore, as per formula (1),

$$y = 0{\cdot}36 \times 3\tfrac{1}{2}'' = 1''{\cdot}26$$

The lever arm z is given by the formula No. (2).

We have therefore

$$z = 0{\cdot}88 \times 3{\cdot}5 = 3''{\cdot}08$$

The length z measured in feet is

$$z = \frac{3{\cdot}08}{12}$$

and the force of the bending couple is

$$F = \frac{M}{z} = \frac{1038 \times 12}{3{\cdot}08}$$

Therefore

$$F = 4044 \text{ lbs.}$$

The working stress of the steel per sq. in. is then

$$t = \frac{4044}{0{\cdot}286} = 14140 \text{ lbs.}$$

Concerning the concrete working in compression, we have a section of compressed concrete per foot run equal to

$$12y = 12 \times 1{\cdot}26 = 15{\cdot}12 \text{ sq. in.}$$

The average working stress of the concrete is therefore

$$\frac{c}{2} = \frac{4044}{15{\cdot}12} = 267{\cdot}46 \text{ lbs.}$$

The stress of the concrete at the extreme compressed fibre is therefore

$$c = 2 \times 267{\cdot}46 \times 2 = 534{\cdot}92 \text{ lbs.}$$

Beam *A*.—We have found for this beam in the preceding article the following data:—

Scantlings	6 in. × 11 in.
Maximum bending moment	38,397 ft.-lb.
Steel in tension	2·53 sq. in.

The thickness of the slab supported by this beam being $4\frac{1}{2}$ in., and the depth of beam measured from the soffit of the slab being 11 in., the table of lever arms shown above gives us

$$z = 0'{\cdot}949$$

Therefore the force of the bending couple is

$$F = \frac{38397}{0{\cdot}949} = 40461 \text{ lbs.}$$

The stress in tension of the steel is therefore

$$= \frac{40461}{2{\cdot}53} = 15992 \text{ lbs.}$$

Concerning the compression of the extreme fibre of the concrete, in order to find this value we shall calculate first the position of the neutral axis given by formula No. (1), namely :—

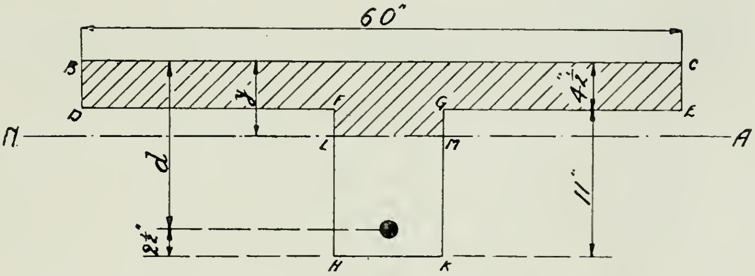
$$y = 0{\cdot}36d$$

$$\text{or}\quad d = 11'' + 4\tfrac{1}{2}'' - 2\tfrac{1}{2}'' = 13''$$

$$y = 0{\cdot}36 \times 13'' = 4''{\cdot}68$$

As the width of the slab working in compression with the beam is equal to 60 in. (see article in number of October), we dispose of a section of concrete working in compression equal to

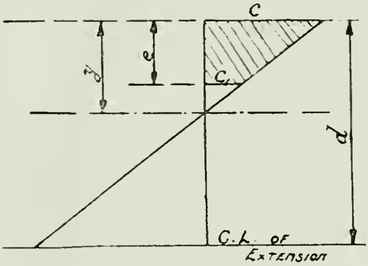
$$4{\cdot}5 \times 60 = 270 \text{ sq. in.} \quad \text{(See } \textit{Fig. } 1.)$$



Note that we have neglected to take into account the area of the rectangle *FGML* in estimating the compressed section, in order to render the calculations easier. The average stress of the concrete in compression is therefore

$$\frac{c+c'}{2} = \frac{40461}{270} = 149{\cdot}86 \text{ lb.}$$

On the other hand, the diagram of deformation, *Fig.* 2, gives us



$$\frac{c}{c'} = \frac{y}{y-e}$$

Hence $$\frac{c}{c+c'} = \frac{y}{2y-e}$$

and $$c = (c+c') \times \frac{y}{2y-e}$$

Substituting $c+c^1$, y and e by their respective values, we have

$$c = 2 \times 149{\cdot}86 \times \frac{4{\cdot}68}{2 \times 4{\cdot}68 - 4{\cdot}5} = 288{\cdot}62 \text{ lb.}$$

If we had found for c a value above 600 lb. this would have indicated that the concrete in compression was insufficient. It would then be necessary either to increase the height of the beam (depth of same measured from underneath the slab) in order to reduce the force of the bending couple or to provide a section of steel in the upper portion of the beam sufficient to absorb the excess of the compressive force above the permissible working stress of 600 lb.

Notice.—We have found, by the first method of verification in the floor slab,

$$c = 534 \text{ lb. and } t = 14160 \text{ lb.}$$

whereas by the second method we have found

$$c = 534{\cdot}92 \text{ lb. and } t = 14140 \text{ lb.}$$

For the beam *a* we have found

1st Method.—$c = 374$ lb. and $t = 15381$ lb.

2nd Method.—$c = 288{\cdot}62$ lb. and $t = 15992$ lb.

We see, therefore, by the above comparison, that for floor slabs the results of the two methods of verification are almost similar. This is because in floor slabs the extreme compressed fibre is usually working at a rate which is not far from 600 lb. per sq. in.

Therefore the position of the neutral axis given by the formula $y = 0{\cdot}36d$ is comparatively accurate.

This position of the neutral axis would be absolutely accurate if the rate of working of the most compressed fibre of the floor slab was equal to 600 lb. per sq. in.

As far as the beams are concerned, the difference between the two results is more accentuated. Here, the compressive stress of the extreme compressed fibre being rather far from the permissible limit of 600 lb. per sq. in., the position of the neutral axis $y = 0{\cdot}36d$ is no longer accurate.

For instance, we have found, by the first method,

$$y = 3''{\cdot}47$$

whereas, by the second method,

$$y = 4''{\cdot}68$$

Nevertheless, the results given by this second method are sufficient for the purpose, and this method is quite safe in its application, as it gives for a section of steel higher stresses per sq. in. than the first method. It follows that by the application of this method of verification the verifier will require a slightly greater section of steel in tension and consequently a great stability will be ensured for the beam under consideration, which is only erring on the right side.

On the other hand, as far as the concrete is concerned, by the application of the second method of verification, we find that the rate of compression per square inch is smaller than the one obtained by the application of the first method, but, as usually this working stress *c* is rather far from the limit of 600 lb., there is nothing to fear concerning the stability of the construction.

In the particular case where the working stress *c* of the concrete at the most compressed fibre becomes equal to 600 lb. the results of the second method are then as accurate as those of the first one, because in that case the formula $y = 0{\cdot}36d$ is absolutely accurate.

The length of this article does not allow me to give at present the method of verification of the portions of the structure resisting the efforts of shearing and of negative bending moments over the supports.

It is my intention, however, to set forth the methods of verification of these particular parts in a further article.

NEW PREMISES FOR MESSRS. H. J. NICOLL & CO., REGENT STREET, LONDON, S.W.

Attention has of late been so closely centered upon purely reinforced concrete construction that we would remind our readers that the interesting development of steel frame building during the last year or two should not be overlooked, inasmuch as this method of construction with concrete surfaces is sure to play an important part in the development of buildings of the future.—ED.

CONSTRUCTIONAL STEELWORK.

THESE important premises form one portion of a large block in Regent Street which is to be erected from the designs of Mr. Henry Tanner, F.R.I.B.A., and although a small part of the work has only been completed up to the present a good idea can be obtained of the whole scheme. The work to the front of Messrs. Nicoll's premises which has been executed in the first instance was carried out while the business of the firm was conducted at the rear of the premises, with temporary entrances and show-cases in Regent Street, and this naturally complicated the work to a certain extent.

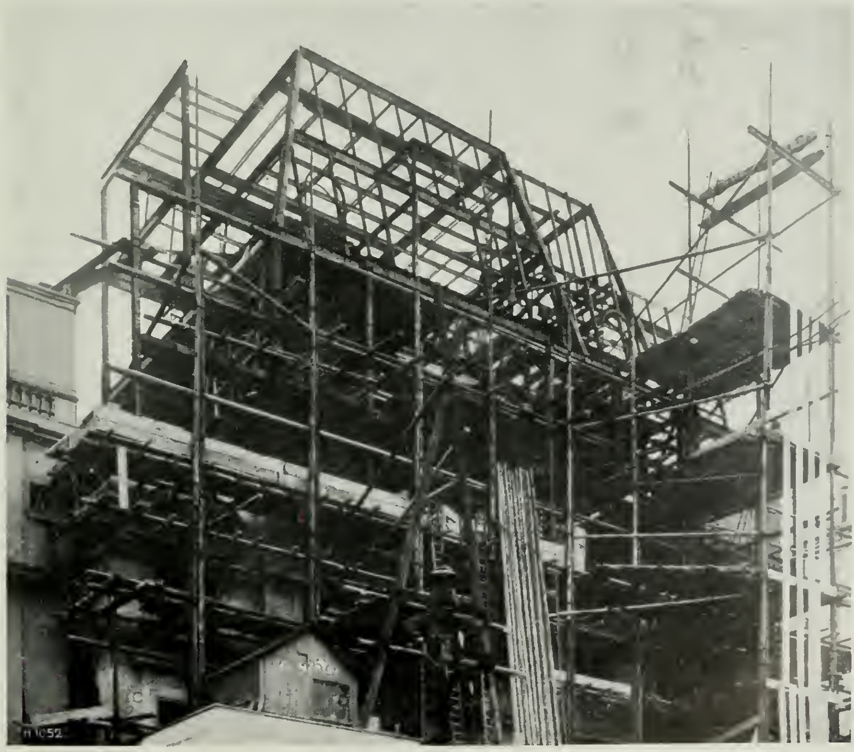


Fig. 1. General View of Steelwork.

The building, which extends right through to Warwick Street, has a frontage to Regent Street of about 75 ft., while the maximum depth is about 106 ft. and of this portion the piece adjoining Regent Street for a depth of about 70 ft. has been executed and the remaining portion is now in hand.

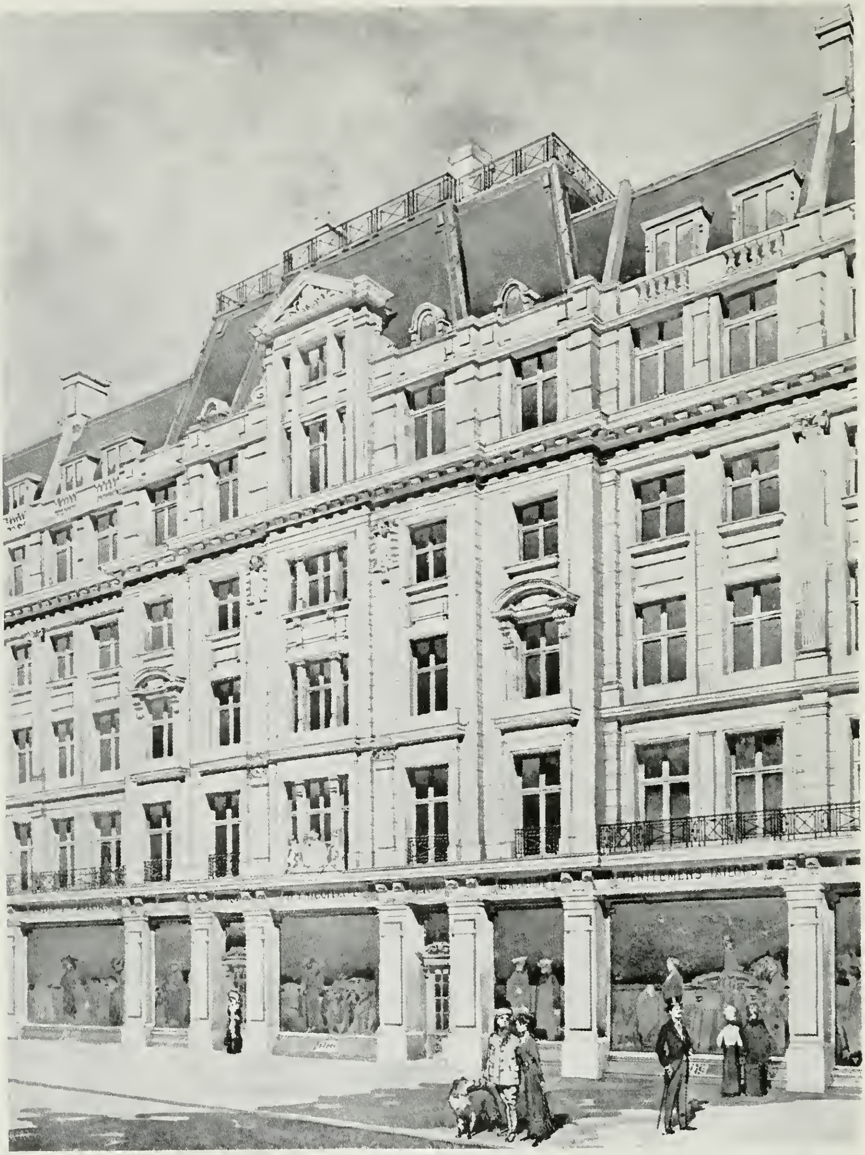


Fig. 2. Front Elevation.
New Premises for Messrs. H. J. Nicoll & Co., Regent Street, London, S.W.

There are six floors above the ground floor and a basement below this, giving a total height from the roof to the lowest floor level of about 95 ft., while the two topmost floors are formed in the roof. The whole of the building is of the steel-framed type, and the general view in *Fig.* 1, showing the upper steelwork in position, will give some idea of the large amount required in the construction.

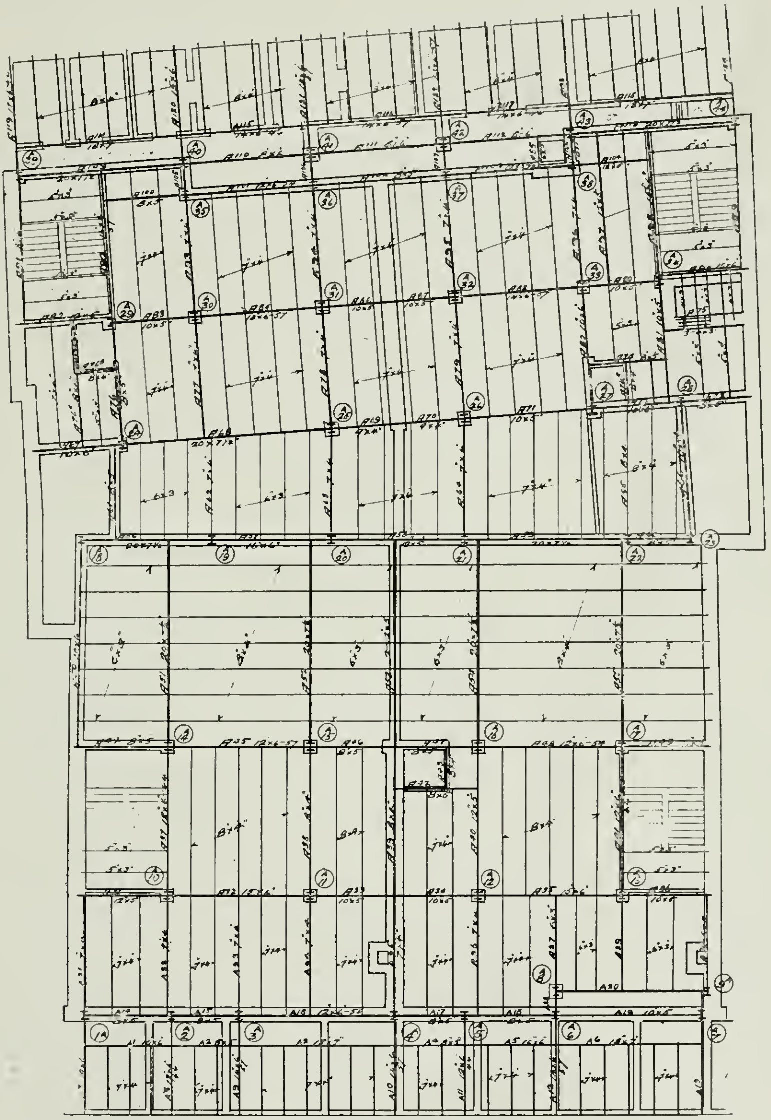


Fig. 3. Basement Plan, showing Ground Floor Steelwork.
New Premises for Messrs. H. J. Nicoll & Co., Regent Street, London, S.W.

There are seven stanchions, spaced at varying distances along the front, to suit the architectural design of the elevation, and owing to one portion of the building projecting forward 9 in. from the general line, with again another set back of 2 ft., they are not all in the same straight line. The stanchions in

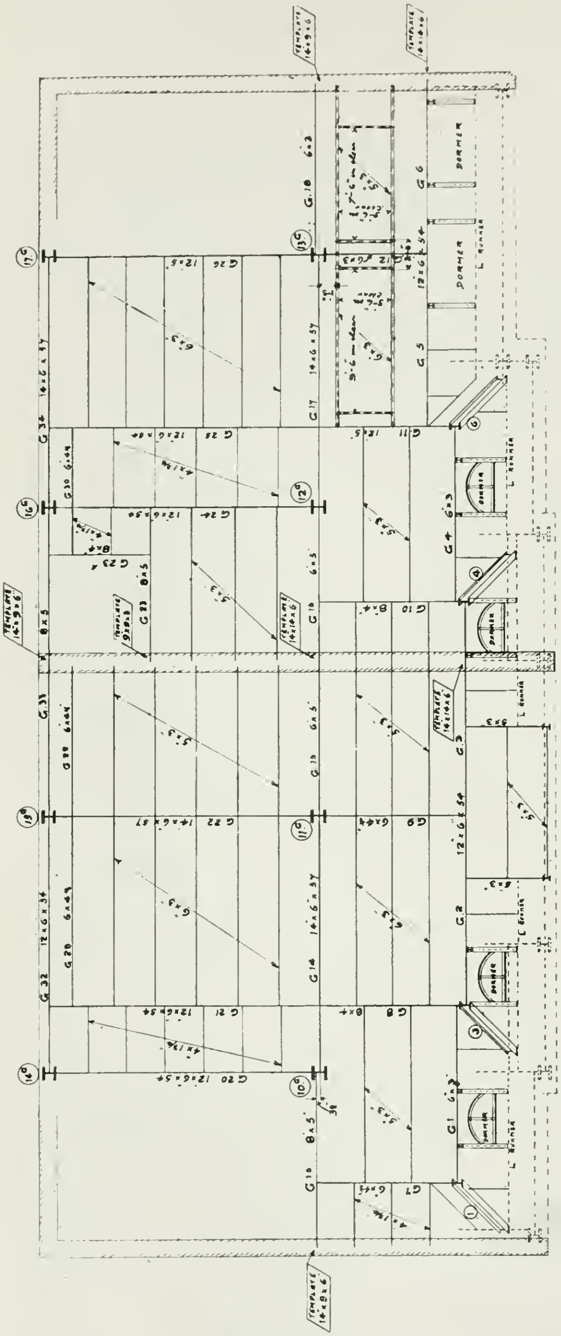


Fig. 4. Fifth Floor Plan, showing Sixth Floor Steelwork.

New Premises for Messrs. H. J. Nicoll & Co., Regent Street, London, S.W.

the interior are spaced also at varying distances, the maximum span being about 25 ft., thus avoiding any large compound girders. All the stanchions are carried on grillage foundations, the largest being 7 ft. square with a depth of 5 ft. 6 in. below the basement floor line. These are constructed with a top layer of three 14-in. by 6-in. by 46-lb. R.S.J.'s, and a bottom layer of eight 8-in. by 5-in. by 28-lb. R.S.J.'s, the latter having a depth of 18 in. of cement concrete below same, and the whole of the joists and the stanchion base were well protected with concrete.

The total number of grillages which were put in under the first part of the scheme amounted to twenty-one, leaving seventeen to be constructed with the rear part of the building.

A typical detail of the type of stanchion employed is illustrated in *Fig.* 5, where the details of several connections are given. It will be seen that the compound stanchions were built up of rolled steel H sections, with plates riveted to the flanges as required. The stanchion illustrated is composed of one 10-in. by 8-in. R.S.J., two 10-in. by ½-in. plates and two 10-in. by ⅜-in. plates, giving a total size of only 11¾ in. by 10 in. The rivets, generally speaking,

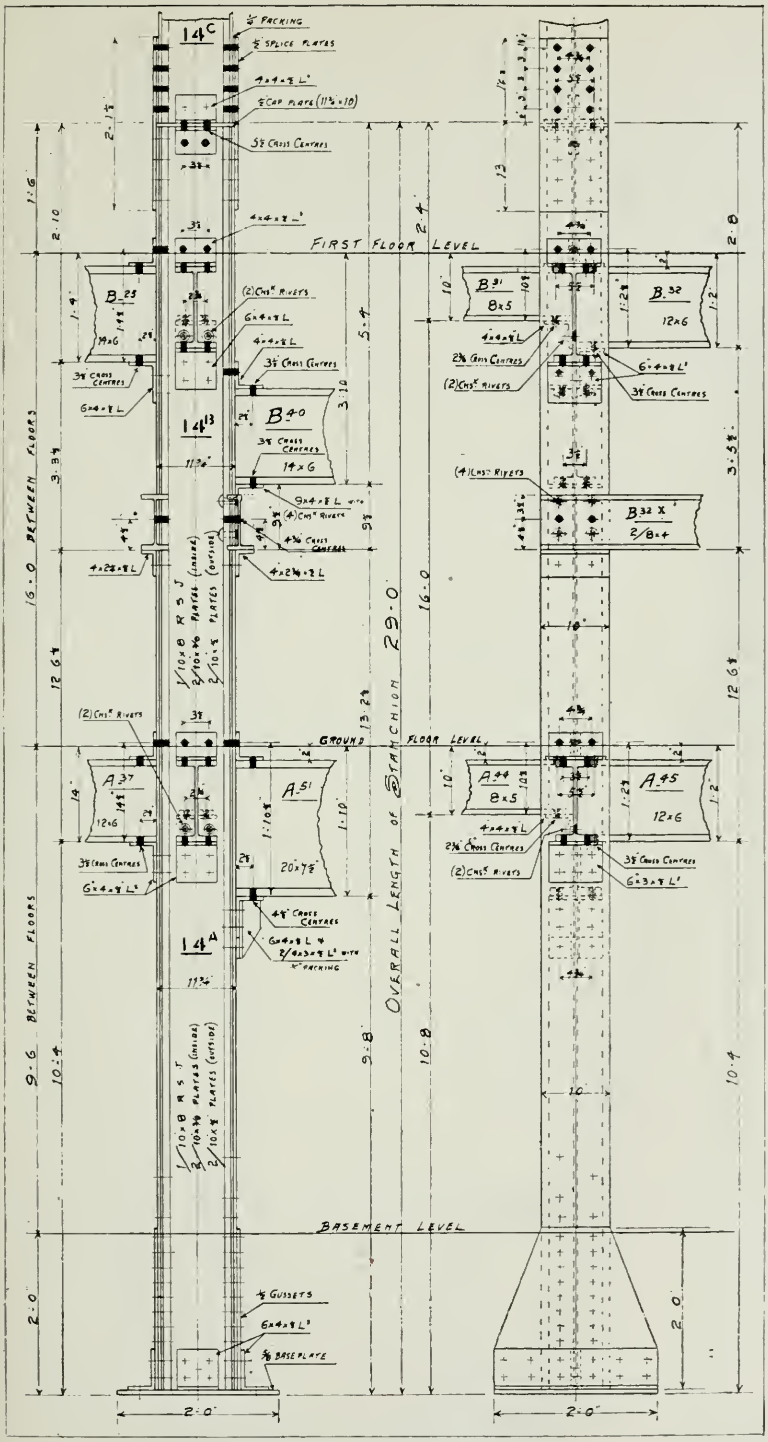


Fig. 5. Typical Stanchion Details.

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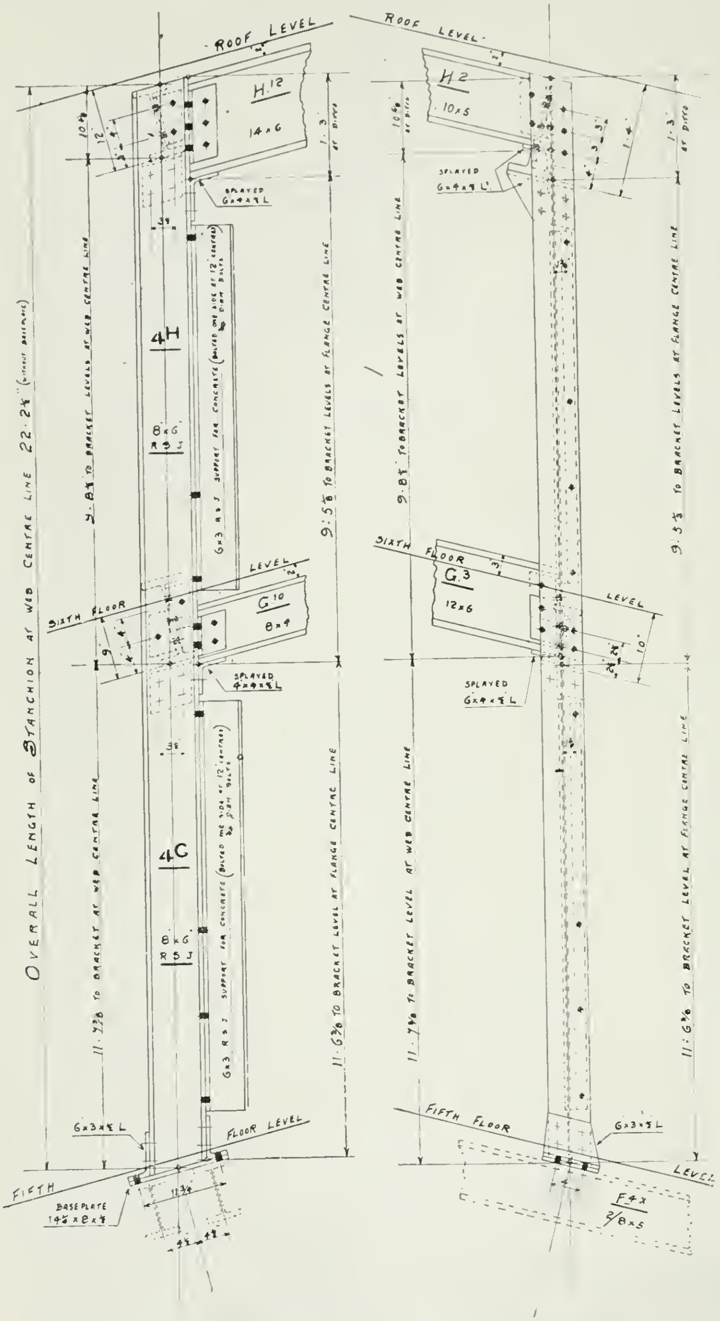
were ⅞ in. in diameter, with a 6-in. pitch. The base was formed with a ⅝-in. plate, 2 ft. square, and ½-in. gusset plates, 2 ft. deep, with 6-in. by 4-in. by ½-in. angle cleats, to form the connections, and wherever possible the stanchions were carried through two floors in one length and connected to the upper length with ½-in. splice plates, about 2 ft. long, in addition to the cap plate and angle cleats riveted in the webs.

The roof rakes illustrated in the detail drawings shown in *Fig.* 6 are interesting, as they occurred at the angles of the roof and thus were required to slope both from the front of the building and also inward toward the centre line of the frontage. The overall length was about 22 ft. and they were composed of an 8-in. by 6-in. R.S.J., with a 14¼-in. by 8-in. by ½-in. base plate, for bolting to the girders upon which they were carried. It will be understood that the connections were rendered somewhat difficult on account of the double slope and the details will give an idea of the methods adopted. The floors throughout were designed to carry a live load of 112 lb. and a dead load of 84 lb. per ft. super and constructed with main beams and filling-in joists with



ROOF LEVEL
H.12
14×6
H2
10×5
SPLAYED 6×4×⅜ L
SPLAYED 6×4×⅜ L
4H
8×6 R.S.J
6×3 R.S.J SUPPORT FOR CONCRETE
OVERALL LENGTH OF STANCHION AT WEB CENTRE LINE 22'·2¼"
9'·8½ TO BRACKET LEVELS AT WEB CENTRE LINE
9'·5⅝ TO BRACKET LEVELS AT FLANGE CENTRE LINE
SIXTH FLOOR LEVEL
G.10
8×4
G.3
12×6
SPLAYED 4×4×⅜ L
SPLAYED 6×4×⅜ L
4C
8×6 R.S.J
11'·7⅞ TO BRACKET AT WEB CENTRE LINE
11'·6⅞ TO BRACKET LEVEL AT FLANGE CENTRE LINE
6×3×⅜ L
FIFTH FLOOR LEVEL
BASE PLATE 14¼×8×½
11¾
F4×2/8×5

Fig. 6. Details of Roof Rakes.
New Premises for Messrs. H. J. Nicoll & Co., Regent Street, London, S.W.

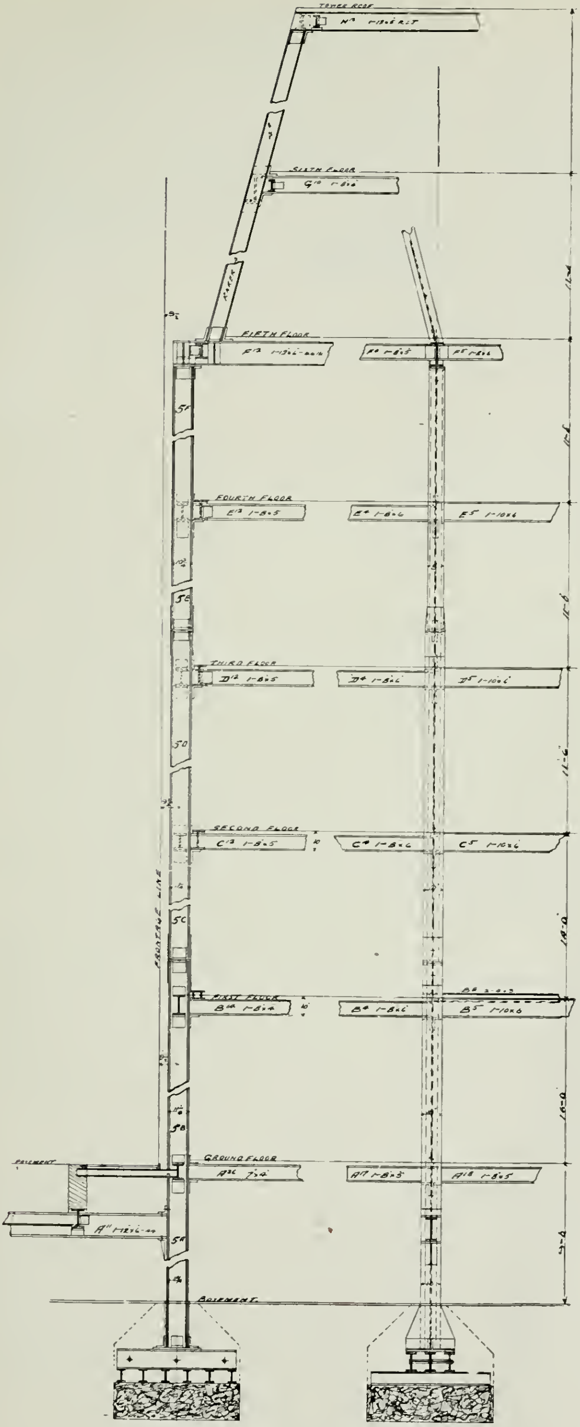


Fig. 7. General Section of Steelwork.
New Premises for Messrs. H. J. Nicoll & Co., Regent Street, London, S.W.

cement concrete between the fillers. The largest main beams were 20 in. by $7\frac{1}{2}$ in. R.S.J. and the filling-in joists were generally 7 in. by 4 in. or 8 in. by 4 in. spaced 3 ft. apart in the clear.

The drawing illustrated in *Fig.* 8 shows the method of framing for the dormer windows in the main roof, and it will be seen that in some cases the semi-circular head was formed by bending angle steels to the required shape and riveting same to the 5-in. by 3-in. rakes which carried the roof concrete. The sill was carried by a 3-in. by 3-in. by $\frac{3}{8}$-in. angle riveted to small angle brackets, also connected to the 5-in. by 3-in. rakes.

With regard to the general plan of the building, excellent accommodation is provided throughout, the various show-rooms and fitting-rooms being exceptionally commodious, while the appearance and importance of the shop on the ground floor is improved by the introduction of a

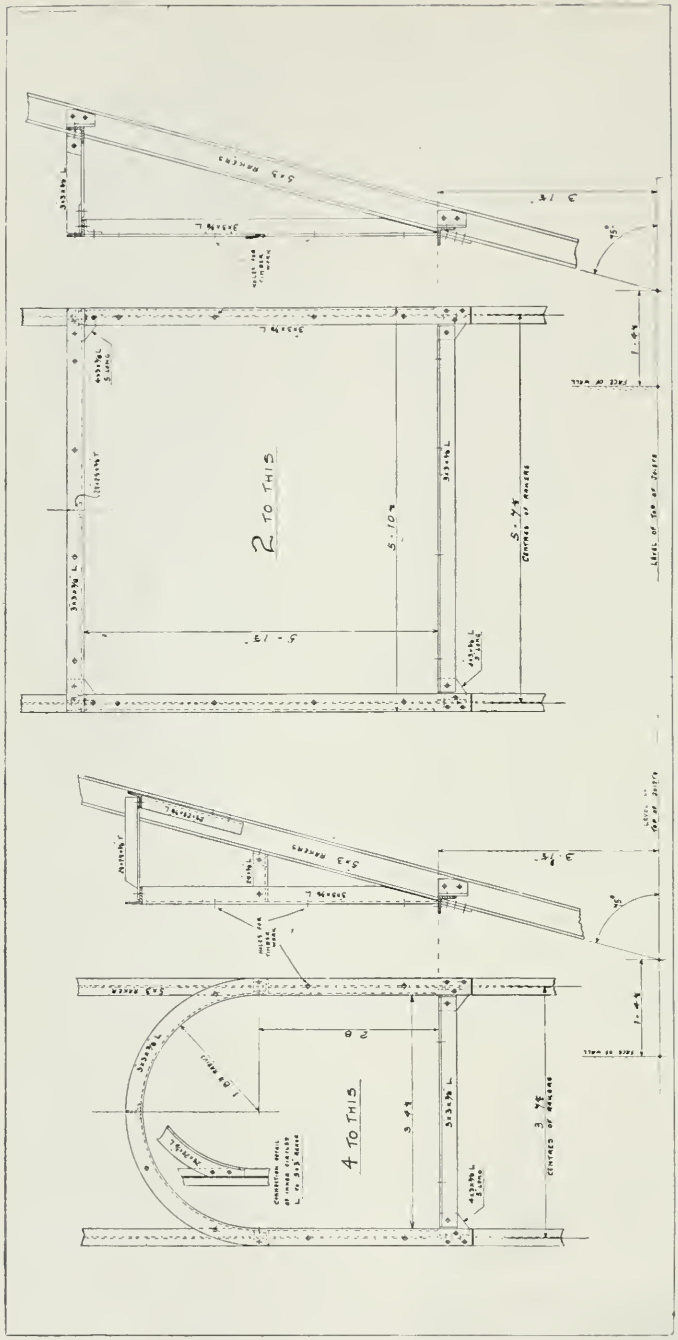


Fig. 8. Steel Framing for Dormers.

New Premises for Messrs. H. J. Nicoll & Co., Regent Street, London, S.W.

gallery at the first-floor level. Owing to the large cubical extent of the premises, it was necessary to introduce party walls and iron doors to divide same up into four buildings, each of which has a staircase extending the full height of the building.

The elevation to Regent Street, which is illustrated in *Fig.* 2, is built in Portland stone, with a slated roof, and has a quiet, dignified treatment which renders it a worthy addition to the architecture of such a fine street.

The contractors for the work were Messrs. Holland & Hannen, and the steelwork was carried out by Messrs. H. Young & Co., of Nine Elms.

THE STORAGE OF PORTLAND CEMENT.

SUGGESTIONS FOR DEALERS, CONTRACTORS AND USERS.

The following advice as to the storage of Portland cement was contained in one of the very useful pamphlets issued by the Association of American Portland Cement Manufacturers, and we commend its perusal to our readers.—ED.

Portland cement is easily affected by moisture. It is purposely made so; for, combined with water and other substances, it forms the best of all building materials—everlasting concrete. But moisture must be kept away from cement until it is needed for actual use. This means that cement must be stored in places which are and can be kept absolutely dry. Upon the proper storage of cement are dependent the reputation of the contractor or builder, the trade of the local dealer, and the satisfaction of the user.

TEMPORARY SHEDS FOR CONTRACTORS AND BUILDERS.

On account of the quantity which they keep on hand, dealers and contractors often build special houses for the storage of Portland cement. Almost any material will do for the construction provided it will shed water and will remain weatherproof. Frequently for temporary storage contractors build sheds of "up and down" plank covered on the outside with one or more thicknesses of tarred paper. The floor must be above possible high water and open so that the wind can circulate under it. To make the house more completely damp-proof, the floor should be double or its equivalent. Often the same effect is obtained by a make-shift double floor of loose timbers and boards laid upon the regular floor. For such a temporary shed there is nothing better than a well-made one-way-slope tarred paper roof on sheathing laid with tight joints lengthwise or in clapboard fashion crosswise. Let the roof have a drip or over-hang of at least a foot on all sides. See that it is absolutely water-tight, that the rain cannot beat in under the eaves, and that the roof is fixed so firmly that a wind-storm will not raise it. It is advisable to put no windows in such a house and to have the door so securely hinged and fastened as to keep out intruders. Where use of cement in the work is dependent upon acceptance by test, provision for piling should be made in accordance with instructions for dealers given below.

STORAGE HOUSES FOR DEALERS.

The same general principles stated above apply to storage houses for dealers. Naturally the house is made more permanent in character. Consequently, it should be supported by a concrete foundation extending into the ground below the frost line. Likewise for a permanent building there is no material so good as well-proportioned concrete. Place the concrete floor on sufficient coarse, compacted gravel or broken stone, fill to bring the floor level a foot above the surrounding ground. Slope the surface of the floor toward the door. For a one-storey building a 6-in. reinforced wall is strong enough. With a reinforced concrete roof, the cement will be protected from all possible danger of dampness.

Since Portland cement weighs practically 100 pounds per cu. ft., the beams of the floors elevated above ground must be heavy and be supported at frequent intervals by concrete piers. If window openings are necessary, do not use sash. A strong door, hinged at the top and capable of being fastened on the inside, is much better for keeping out dampness. Often the building has a solid or skeleton lining on the inside, for reasons given under directions for storing cement. It should be well sway-braced on the inside to prevent springing of cracks or bulging.

The size of the house is dependent upon the extent and character of the dealer's trade. An average carload of Portland cement contains about 175 to 200 barrels of four bags each. In determining the necessary size of the cement sheds, consider that each bag of cement stored will occupy one cu. ft. Do not build too small. There is an increasing demand everywhere for Portland cement and the trade of local dealers is far surpassing expectations.

The character of the trade has much to do with fixing the size of the storage house. Especially dealers who supply contractors should have sufficient room to keep every shipment separate; for cement used by contractors and builders must often pass a seven- or twenty-eight-day test before being used. Moreover, every cement storage house should be large enough to have aisles between the piles of various shipments, so that cement may be removed from storage in the same rotation as it was put in.

Cement on the Work, Piled Properly on Wooden Platform.

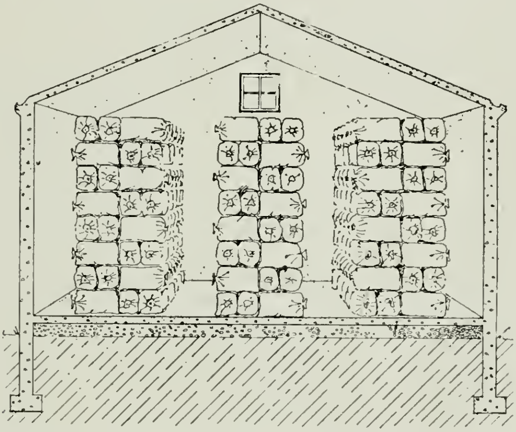
STORAGE FOR USERS OF CEMENT.

Many users of Portland cement need to store it only for the short interval between the time of hauling it from the dealer and using it in the concrete work. Any building with a good roof, tight side-walls, and a dry floor will do. Make a temporary double floor by means of small beams or logs and loose boards. See that driving rains or damp air cannot reach the cement.

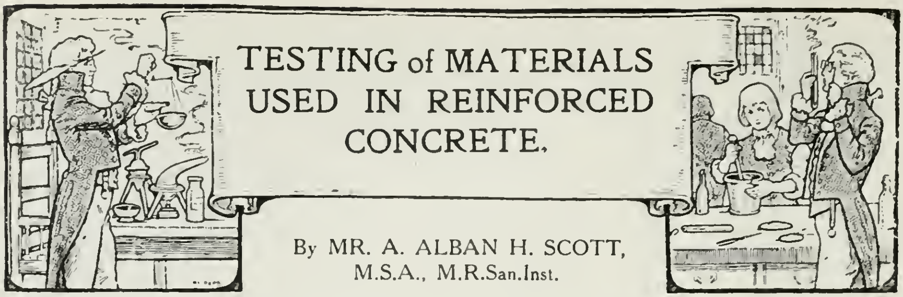
DIRECTIONS FOR STORING CEMENT.

Portland cement weighs nearly 100 pounds per cu. ft., therefore judgment must be exercised in loading the floor. Likewise, on account of its weight and possible damage from dampness, do not pile the cement against the side-walls of the building unless the house is full-lined or skeleton-lined on the inside. Store the cement so as to leave an open space or an aisle along the side-walls and aisles at places necessary to separate shipments. Get rid of all possibility of the cement piles toppling over by laying the bags, as a mason would say, "headers and stretchers," that is, alternately lengthwise and crosswise, so as to tie the pile together. Examine the building frequently to see that there are no leaks in the roof or side-walls.

Even on the work never pile cement on the ground or on bricks. Throw down a few blocks, lay boards upon them and make a dry floor for the cement. Have just enough cement on the work to keep things going. A good tarpaulin is handy to cover up the cement in case of a sudden shower. Always buy cement from the dealer who has a dry store-house. If the cement in the bag has been wet it is hard as a rock: do not use it under any circumstances. However, do not mistake lumps caused by pressure in the store house for "set-up" cement. Such lumps crumble easily and the cement is perfectly good. Take care of the Portland cement just as carefully after receiving it as a good dealer does, and, properly mixed with sand and stone or gravel, it will yield a building material proof for all time against fire and repairs.



Cross Section of Concrete Storage House, with Cement Correctly Piled.



TESTING of MATERIALS USED IN REINFORCED CONCRETE.

By MR. A. ALBAN H. SCOTT,
M.S.A., M.R.San.Inst.

The following is the text of a very interesting paper recently read by Mr. A. Alban H. Scott before the Society of Architects.—ED.

Concrete and reinforced concrete are now being used so extensively and for such a variety of purposes, and its possibilities are so expansive, that it requires at the present stage some further careful investigation by architects, both as to the strength and properties of the various units, as well as the finished material.

The necessity for some very serious consideration has been particularly impressed upon my mind, in view of the fact that certain figures are being laid down as the ultimate strength of concrete by the various reports and regulations issued comparatively recently.

These figures seem to have been based upon results obtained from laboratory-made specimens only, made under the most favourable conditions, without any allowance for the more or less rough methods which are only obtainable at present on the actual construction. In making some of the laboratory test specimens the materials are very accurately gauged, thoroughly and evenly mixed, and in lieu of the ordinary punning and slight ramming (if any) which the materials get on the works, they are subjected to pounding down with a heavy hammer, thus getting an artificial result, such as it is impossible to attain under the most perfect conditions obtainable on even a perfectly organised job.

The test results to be considered this evening are such as can be reasonably expected from work executed under a specification such as the author published last year; but even these results can only be expected if professional supervision is given, not only to the general work but also to the most minute points.

It has always seemed to me to be inconsistent to take elaborate precautions to obtain cement of a high and even quality and treat the other component parts as if the ultimate strength of the concrete did not also rely upon their influence.

The following is an outline procedure I would suggest should generally be adopted with regard to the testing of materials for reinforced concrete work.

All tests to be made at an independent testing and experimental works.

Testing of Cement.—The cement to be tested from samples taken from the bulk, such samples being taken from various parts of the bins at the makers' works. After the cement has arrived on the job, samples again taken from various bags, thoroughly mixed and again tested. Further tests made from time to time as the material is being used in the works, the number of these tests being regulated by the magnitude of the consignment, and the time taken in using same.

All tests of cement should be made in accordance with the British Standard Specification for Portland Cement, with the *further* test for ascertaining resistance to thrusting stress of both neat cement and also cement and sand, in the same proportions used for the tensional briquettes.

Aggregate or Coarse Material.—A sample in bulk to be delivered on to the works, a sample taken and tested :—

1st. For freedom from loam and other foreign matter.
2nd. For the proportion of the various sizes of the crushed material.
3rd. For the amount of voids.
4th. For specific gravity.

Tests repeated on the material being used in the works from time to time as may be considered desirable.

Sand.—Sand to be treated in exactly the same way as aggregate, and in addition a test for the amount of material that will pass a sieve of $1/50'' \times 1/50''$ apertures should be ascertained; this, which can be called "flour," should be rejected.

Water is tested to see that it contains no unusual or injurious properties.

Steel to be first inspected at the makers' works with a view to ascertaining :

1st. Whether welds have been made.
2nd. For surface defects.
3rd. Correctness of diameters.

Every rod over $\frac{3}{4}''$ diameter must be stamped with a die *as having been inspected.* Smaller bars may be bundled and tie stamped. Sample lengths are taken from the actual rods, which are duly stamped and sent to the testing works for the purpose of ascertaining :—

1st. Their ultimate strength.
2nd. Their elastic limit.
3rd. Their elongation and contraction of area.
4th. To observe the structure of the metal at fracture—whether the same is silky, granular, or fibrous.
5th. Bending test.

All these tests to apply to rods and wire from and including $3/16''$ diameter and upwards.

Concrete.—The test specimens of concrete should be of a standard size of 6″ cube. Six specimens made for each test, three made in the laboratory and three on the works. The cement for each six sets of specimens to be taken from the same consignment. The laboratory tests specimens should be made as far as possible on practical lines, so that the result should be such as can be reasonably expected from concrete in the actual work.

All specimen pieces made on the works should be made from concrete taken from the actual mixing platform. All such specimens should be made in metal moulds, and the concrete worked in by punning and tamping to the same degree as has actually taken place in the structure. Six test cubes should be used for each test, and the tests should be made at the following minimum periods :—

7 days.
28 ,,
56 ,,
90 ,,
and 1 year.

For the purpose of record and research work such tests should be carried out at the following periods :—

7 days.
28 ,,
56 ,,
90 ,,

6 months.
9 ,,
12 ,,

2 years.
3 ,,
4 ,,
and 5 ,,

Centering.—No mechanical tests are required for this, but when the strutting is being placed into position rough calculations should be made to see if any undue deflection or movement is likely to take place during the process of placing the moist concrete into position, and inspection made to see that the joints have been properly filled up by rubbing with hard bar soap or other material to prevent dripping of the cement and sand.

In reinforced concrete the concrete takes compression and shear; the steel taking *all* the tension and assisting for horizontal and vertical shear and compressional strains. The concrete being stressed to not more than 600 lb. per sq. in. in compression and 60 lb. per sq. in. in shear; the steel in compression in beams not more than nine times per sq. in. that of the adjoining concrete, and in columns not more than fifteen times that of the adjoining concrete and steel in tension 15,000 lb. per sq. in. and the adhesion of the concrete to the steel 100 lb. per sq. in. of actual contact.

These figures being subject to various conditions, such as the efficient tying in of the steel in compression to prevent bulging of the rods, proper proportion of metal to the concrete and spacing of the steel. In the case of columns the effective area of concrete is only taken as that part which is hooped in by the ties to the vertical steel members.

If we are working to a factor of safety of a fourth of the ultimate, the ultimate resistance to thrusting stress of the concrete should be not less than 2,400 lb. per sq. in., the resistance of concrete to shear 240 lb. per sq. in., and the steel 60,000 lb. per sq. in. ultimate tensile stress.

These figures show that all the materials are called upon to be of high efficiency, and to work in harmony and simultaneously together, and have been adopted in consideration not only of their ultimate resistance but also the relative coefficient of elasticity.

The work is designed on the assumption that *the materials actually used in the construction* are capable of resisting these stresses.

The following are a few reasons why the author considers the tests necessary:—

Aggregate and Coarse Material.—It has been found that most aggregates (unless washed) contain loam and other foreign matter. A sample of river aggregate recently tested gave as much as 7 per cent. of loam, and most loams have a surprising covering power, being of the very finest of "flour," and consequently when the cement is added there is a thin film between the cement and the actual siliceous material, thus preventing the cementing together of the particles.

Some time ago it was advocated in certain quarters that the presence of loam in aggregate did not decrease, but rather increased, the strength of concrete; a slight mistake was made there, the material which was mistaken for "loam" actually contained certain cementing properties which had the effect of making the concrete a richer mixture.

With aggregates which are practically uniform in size, or if the various sizes are not properly graded, proper bond is not obtained between the various materials as the mortar is concentrated, and thus a portion of the mixture is deficient in cementing materials, therefore it is most desirable that the grading and voids should be worked out very carefully, and, with certain exceptions, it is found, within limits, that the grading which gives the smallest amount of voids in the aggregate and sand results in concrete of greater strength. With aggregate and sand containing a high percentage of voids a greater proportion of cement is required.

TABLE I.
Results of Tests of Aggregates.

	Size square apertures	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{1}{8}$	$\frac{1}{26}$	$\frac{1}{32}$	$\frac{1}{50}$	Not graded
	Parts	I	I	I	I	I	I	I	I	I	
1. Thames Sand...	Perc. of voids	—	—	—	—	39·9	39·0	35·5	34·5	—	—
2. Sea sand on shore 26 years		—	—	—	—	26·7	29·5	32·0	—	—	—
3. Ordinary Thames Sand ...		—	—	—	—	—	—	—	—	—	23·3
4. River sand ...		—	—	—	—	—	—	—	—	—	32·0
Sand from crushed rock		—	—	—	—	—	—	—	—	—	33·2
5. Crushed rock ...		40·1	—	44·1	—	—	—	—	—	—	—
6. Thames ballast graded ...		48·2	44·5	43·5	42·5	—	—	—	—	—	—
7. Sea ballast ...		41·8	42·3	41·8	42·0	—	—	—	—	—	—
8. Ballast and sand as from Banks to pass only, and not retained on apertures ...		37·0	38·0	37·6	35·6	—	—	—	—	—	—
10. Thames sand and ballast not crushed...	Parts	—	—	—	—	I	I	2	2		
		—	—	—	—		32·0				
,, ,,		—	—	—	—		28·7				
	Parts	I	2	3	4	I	I	2	2		
,, ,,			42·3								
,, ,,					29·3						
,, ,,					25·1						
11. Sea Sand and ballast	Parts	I	I	4½	3½	I¼	3¾				
				29·6							
,, ,,	Parts	I	I	4½	3½	I½	I½	3			
					23·9						
12. Standard sand passing 20 × 20 retained on 30 × 30 mesh sieve ...			34·5								

In this connection it should be remembered that as the number of particles increase so the proportion of the cement to the whole should be also increased, owing to the greater covering power required. A test cube made with a very small amount of aggregate and a large amount of sand with the usual proportions of cement gave an ultimate resistance to thrusting stress of only about 600 lb. per sq. in.; such low result was entirely owing to the fact that the usual amount of cement in that case was not sufficient to properly cover each particle of the sand and aggregate, and a perfectly cemented and homogeneous mass was not obtained.

The ultimate resistance of any material is its strength at its weakest point, and the actual area of the concrete is the gross area minus the area of voids at any section; it is therefore necessary that the ascertained percentage of voids in the aggregate and sand should be entirely filled up with cement with an additional allowance for completely surrounding each particle. The usual rough and ready means of determining the "exact" proportion of the cement to the sand and aggregate have resulted in many cases of disaster.

The specific gravity of sand and cement should be taken in order to compare with the weight of the test specimens, as such weight is materially affected by the original weight of the aggregate and sand. There are at least two methods of ascertaining the amount of voids, one is by ascertaining the specific gravity of the material used, and, secondly, by allowing the aggregate and sand to absorb moisture, then to dry the surfaces without extracting moisture from the material, and then in the test tubes to add water until such time as the level of the aggregate and the water is at the same point.

Water.—The water usually obtained from town supplies can be taken as of proper quality for use in concrete work, but in country districts water obtained from wells and reservoirs often contains foreign matter, such as peat and other vegetables, and it has been found that the use of peaty water retards the setting of the cement; in one case the concrete did not set for a period of three weeks, at which stage the concrete was still very soft and was taken up.

Steel.—In no part of the steel for reinforced concrete should welds be allowed. Welds can be made in various ways, but it is impossible to test each weld, and in these joints it is not an exaggeration to say that not one weld in 500 would be of equal strength to the rest of the bar, and it has been ascertained, with most disastrous results, that the strength at the point of the weld goes so low as 30 per cent. of the bar adjoining. This is caused not so much by the lack of amalgamation of the material itself, but generally by the fact that large voids are left right in the centre of the joint.

Surface defects in steel are often found in the ordinary commercial bars, being indicated by most minute cracks, generally starting in the shape of a "V." Although such defects are in themselves perhaps almost innocent, yet immediately the material is subject to any stress they develop in a most alarming manner.

It has been found that the diameters of bars vary from that specified, resulting in one case in a loss of 12 per cent. in the *area*. Excess diameter is also often met with, and as the final measurements are taken on a basis of the correct diameter such excess of diameter will often be the solution of the difference found between the surveyor's measurements and the weight alleged to have been used by the contractor. Reinforced concrete work being designed on the assumption that the whole of the materials employed are exactly in accordance with the specification, a reduction in area of the bars is most serious.

The ultimate strength should be taken at not less than 60,000 lb., and not more than 72,000 lb. per sq. in.

It is necessary to keep a very careful watch upon every piece of steel that is brought on to the site.

The elastic limit is also of the utmost importance, as although steel might have a high ultimate resistance, the elastic limit might be so low that in case an accidental load is placed on the work a sudden collapse might take place; whereas by use of material with a proper elastic limit proper warning would be given before its sudden failure.

It is essential that steel should have such properties as will enable it to take a gradual and uniform extension, thus indicating a uniformity of quality; and the contraction of area at fracture should not be less than 45 per cent. for bars 1″ diameter and under, and 40 per cent. for bars over 1″ diameter.

A silky fracture indicates uniformity and a good mild steel, so far as the metal, as metal, is concerned. Granular grain indicates brittleness and unequal quality.

Concrete.—I think it is safe to say that of all materials used in the building trade concrete is liable to and does vary more than any other material. Even if every care has been taken to see that each unit is of its proper quality and strength, yet we have in the finished concrete to reckon with a very large amount of human element. One gang of men may produce concrete of the best quality, and another gang on the same work with similar materials may give very indifferent results. This depends a very great deal upon the head ganger, and upon the foreman who selects such ganger. With a good machine mixer the possibility of human error is considerably decreased, but at the same time an improperly designed mixer is, in my opinion, more dangerous than mixing the concrete by hand.

It was found that concrete which has its setting somewhat retarded by sprinkling with water gives generally higher results than concrete which was allowed to dry under normal conditions. The question arises here, however, as to the consistency of concrete not only from the point of view of the strength of the concrete itself, but rather as to what consistency will give the best results in actual practice from the point of view of contact with the metal, resistance to crushing, and shear, and although it is essential that only as little water as possible should be used to prevent airholes occurring after the water has been evaporated, I would feel inclined to use a concrete slightly wetter than the average practice in France, but certainly much drier than the general present use in England.

Comparatively wet concrete will not allow of it being rammed or "tamped"; whereas, to get a drier concrete into its correct position, tamping and gentle ramming is essential. As the whole of the strength of reinforced concrete depends upon proper adhesion of the concrete to the steel, it is an essential factor that the concrete is gently rammed at every point.

The architect has therefore a much better chance of making sure that the concrete is properly placed in position and properly worked between the reinforcing metal, and the greatest trouble has been found to be with contractors wishing to make the concrete wetter than is desirable owing to the fact that a wet concrete is more easily placed in position with much less labour than a dry concrete.

The usual period of the first test on concrete has until recently been 28 days, but with important structures, to obtain tests at 28 days after the concrete is made is not going to be of very much use for early correction when the work is being rapidly pushed forward, therefore a seven-day test is essential, so as to be able to immediately detect any error in any of the materials or workmanship.

At the present moment there are few tests at seven days, so that for the present the reasonable resistance of concrete at this period has not been definitely ascertained. (Approximately 350 to 400 lbs. per square inch.) Such tests in connection with the 28 days' tests will also give most useful information with regard to when it is safe to strike the centering, and when one considers the failures that have taken place owing to the centering having been removed too quickly, a seven-days' test will undoubtedly become a recognised factor very shortly.

The 56 days' and 90 days' tests are desirable, as they show what is actually taking place in the work with regard to the increase or decrease in the strength of the structure. The concrete up to about six months of age increases in strength fairly rapidly, but it is curious that from about six to nine months the increasing resistance to thrusting stress is very trifling, but after nine months the strength of the concrete again continues to increase at a more rapid rate, although not so rapid as during the first six months.

TABLE II.

Results of Experiments to Ascertain the Resistance to Thrusting Stress of Sixty-four Concrete Tubes, made under Various Conditions at these Works.

Test No.	Description.		Weight.	Dimensions.	Base Area.	Crushed.				
						Stress.	Per sq. in.		Per sq. ft.	
s s	**Age, 28 days.**		lb.	in.	sq. in.	lb.	lb.		tons.	
2,975	Mixed fairly wet (as in practice) A	Not sprinkled	17·90	6·00 × 6·00 × 6·00	36·00	34,980	972	1,026	62·5	66·0
2,976			18·17	6·02 × 6·00 × 6·00	36·00	36,080	1·002		64·4	
2,977			18·31	6·04 × 6·00 × 6·00	36·00	39,430	1,095		70·4	
2,978			18·16	6·04 × 6·00 × 6·00	36·00	37,350	1,039		66·7	
2,983		Sprinkled with water every other day for first 3 weeks	18·60	6·10 × 6·00 × 6·00	36·00	42,750	1,187	1,118	76·3	71·9
2,984			18·31	6·04 × 6·00 × 6·00	36·00	37,920	1,053		67·7	
2,985			18·43	6·01 × 6·00 × 6·00	36·00	41,500	1,153		74·1	
2,986			18·32	6·02 × 6·00 × 6·00	36·00	38,890	1,080		69.5	
2,991	Mixed fairly dry not (rammed) B	Not sprinkled	18·04	6·01 × 6·00 × 6·00	36·00	67,700	1,881	1,718	121·0	110·5
2,992			18·02	6·00 × 6·00 × 6·00	36·00	64,300	1,786		114·9	
2,993			18·15	6·03 × 6·00 × 6·00	36·00	54,700	1,519		97·7	
2,994			18·34	6·06 × 6·00 × 6·00	36·00	60,700	1,686		108·4	
2,999		Sprinkled	18·22	6·05 × 6·00 × 6·00	36·00	57,300	1,592	1,668	102·4	107·3
3,000			18·26	6·04 × 6·00 × 6·00	36·00	64,600	1,795		115·4	
3,001			18·10	6·00 × 6·00 × 6·00	36·00	58,800	1,633		105·0	
3,002			18·17	6·02 × 6·00 × 6·00	36·00	59,500	1,653		106·3	
3,121	Mixed fairly dry (rammed) C	Not sprinkled	18·36	6·01 × 6·00 × 6·00	36·00	68,000	1,889	2,018	121·5	129·8
3,122			18·58	6·04 × 6·00 × 6·00	36·00	75,000	2,083		133·9	
3,123			18·28	6·03 × 6·00 × 6·00	36·00	73,630	2,045		131·5	
3,124			18·37	6·03 × 6·00 × 6·00	36·00	74,010	2,056		132·2	
3,129		Sprinkled	18·50	6·04 × 6·00 × 6·00	36·00	71,460	1,985	2,032	127·6	130·7
3,130			18·66	6·03 × 6·00 × 6·00	36·00	74,270	2,063		132·7	
3,131			18·56	6·04 × 6·00 × 6·00	36·00	74,860	2,080		133·8	
3,132			18·43	6·02 × 6·00 × 6·00	36·00	72,060	2,002		128·7	
3,007	Mixed very dry (rammed) D	Not sprinkled	18·13	6·03 × 6·00 × 6·00	36·00	80,580	2,238	2,225	143·9	143·0
3,008			18·20	6·02 × 6·00 × 6·00	36·00	83,100	2,308		148·4	
3,009			17·90	6·00 × 6·00 × 6·00	36·00	83,280	2,313		148·7	
3,010			18·05	6·01 × 6·00 × 6·00	36·00	73,470	2,041		131·2	
3,015		Sprinkled	17·72	6·02 × 6·00 × 6·00	36·00	63,250	1,757	2,078	113·0	133·6
3,016			18·28	6·00 × 6·00 × 6·00	36·00	83,460	2,318		149·1	
3,017			18·10	6·00 × 6·00 × 6·00	36·00	85,020	2,362		151·9	
3,018			17·72	6·00 × 6·00 × 6·00	36·00	67,500	1,875		120·6	

TABLE IIa,

Test No.	Description.		Wght.	Dimensions.	Base Area.	Crushed.					Increase over 28 days
						Stress.	Per sq. in.		Per sq. ft.		
s s	**Age, 90 days.**		lb.	in.	sq. in.	lb.	lb.		tons.		%
2,979	Mixed fairly wet (as in practice) A	Not sprinkled	17·76	6·00 × 6·00 × 6·00	36·00	64,720	1,798	1,787	115·6	114·9	74·1
2,980			17·77	6·00 × 6·00 × 6·00	36·00	63,920	1,776		114·2		
2,981			17·78	6·00 × 6·00 × 6·00	36·00	64,560	1,793		115·3		
2,982			17·95	6·00 × 6·00 × 6·00	36·00	64,200	1,783		114·7		
2,987		Sprinkled with water every other day for first 3 weeks	17·57	6·00 × 6·00 × 6·00	36·00	58,190	1,616	1,789	103·9	115·5	60·6
2,988			17·64	6·00 × 6·00 × 6·00	36·00	62,930	1,748		112·4		
2,989			17·82	6·00 × 6·00 × 6·00	36·00	65,130	1,809		116·3		
2,990			17·87	6·00 × 6·00 × 6·00	36·00	71,410	1,984		127·6		
2,995	Mixed fairly dry (not rammed) B	Not sprinkled	18·13	6·00 × 6·00 × 6·00	36·00	87,410	2,428	2,440	156·1	156·9	42·0
2,996			18·08	6·00 × 6·00 × 6·00	36·00	85,960	2,388		153·6		
2,997			18·11	6·01 × 6·00 × 6·00	36·00	89,880	2,497		160·6		
2,998			18·12	6·01 × 6·00 × 6·00	36·00	88,180	2,449		157·5		
3,003		Sprinkled	18·18	6·01 × 6·00 × 6·00	36·00	90,400	2,511	2,514	161·5	161·7	50·7
3,004			18·30	6·01 × 6·00 × 6·00	36·00	86,950	2,415		155·3		
3,005			18·28	6·01 × 6·00 × 6·00	36·00	93,850	2,607		167·7		
3,006			18·25	6·01 × 6·00 × 6·00	36·00	90,800	2,522		162·2		
3,125	Mixed fairly dry (rammed) C	Not sprinkled	18·34	6·00 × 6·00 × 6·00	36·00	100,500	2,792	2,914	179·5	187·4	44·4
3,126			18·45	6·00 × 6·00 × 6·00	36·00	105,100	2,919		187·7		
3,127			18·52	6·00 × 6·00 × 6·00	36·00	106,900	2,969		190·9		
3,128			18·25	6·00 × 6·00 × 6·00	36·00	107,200	2,978		191·5		
3,133		Sprinkled	18·46	6·00 × 6·00 × 6·00	36·00	117,300	3,258	3,159	209·5	203·1	55·4
3,134			18·40	6·00 × 6·00 × 6·00	36·00	109,200	3,033		195·0		
3,135			18·32	6·00 × 6·00 × 6·00	36·00	124,400	3,456		222·2		
3,136			18·36	6·00 × 6·00 × 6·00	36·00	104,000	2,889		185·8		

TABLE IIa. (cont.)

Test No.	Description.		Wght.	Dimensions.	Base Area.	Crushed.					Increase over 28 days.
						Stress.	Per sq. in.		Per sq. ft.		
s s	**Age, 90 days.**		lb.	in.	sq. in.	lb.	lb.		tons.		%
3,011	Mixed very dry (rammed) D	Not sprinkled	18·28	6·01 × 6·00 × 6·00	36·00	128,000	3,556	3,311	228·7	212·9	48·9
3,012			18·00	6·01 × 6·00 × 6·00	36·00	119,020	3,306		212·6		
3,013			18·06	6·01 × 6·00 × 6·00	36·00	121,620	3,378		217·2		
3,014			17·69	6·00 × 6·00 × 6·00	36·00	108,150	3,004		193·2		
3,019		Sprinkled	17·82	6·00 × 6·00 × 6·00	36·00	110,980	3,083	3,554	198·3	228·5	71·0
3,020			17·90	6·00 × 6·00 × 6·00	36·00	130,050	3,612		232·3		
3,021			17·98	6·00 × 6·00 × 6·00	36·00	136,220	3,784		243·3		
3,022			18·10	6·00 × 6·00 × 6·00	36·00	134,500	3,736		240·3		

Proportions : 4 cu. ft. Thames ballast (passing ¾ in. and retained on ¼-in. mesh).
2 cu. ft. sand (25 per cent. passing ¼-in. and retained on ⅛-in. mesh, 75 per cent. passing ⅛-in. mesh).
1 cu. ft. "Ferrocrete" Cement (76·2 lb.)
For tensile strength, etc., of Cement used for above tests, see report dated November 9th, 1910.

Water added to above quantities.		Gals.	Qts.	Pts.	
	Group A	5	1	1	= (53·75 lb.)
	,, B	4	2	0	= (45·0 lb.)
	,, C	4	0	0	= (40·0 lb.)
	,, D	3	2	0	= (35·00 lb.)

Percentage of Voids.

SS4780	Ballast passing ¾-in. and retained on ¼-in. mesh	41·3 per cent.
SS4779	Sand ,, ¼-in. ,, ,, ⅛-in. ,,	40·5 ,, ,,
SS4778	Sand ,, ⅛-in. ,, ,,	29·1 ,, ,,

99 Southwark Street, London, S.E.
December 2nd, 1910.

David Kirkaldy & Son.

You will see that there are many things to be learnt from this excellent series of tests made at Messrs. Kirkaldy's works. At 28 days the highest result obtained was an average of 2,225 lb. per sq. in., and at 90 days the highest result was 3,554 lb. These two results can be taken as being the greatest amount of resistance that can be obtained from concrete made under the most favourable conditions, and it should be compared with the lowest results obtained, which is 1,026 lb. per sq. in. in 28 days, and 1,787 lb. per sq. in. in 90 days, with varying results between these two extremes, and the question arises as to which is the most probable result that would be obtained from work in actual practice.

TABLE III.
Result of Grading of Sand.

Sand.	Retained on 30 × 30	Passed 30 × 30 Retained on 50 × 50	Passed 50 × 50	Total.
Sample 1.	29·0 p.c.	15·5 p.c.	55·5 p.c.	100·0
Sample 2.	48·2 p.c.	14·0 p.c.	36·8 p.c.	100·0

We have heard that in certain concrete works 2,000 lb. per sq. in. is invariably obtained on concrete 28 days old, but I hesitate to accept this figure, and such high results are possibly accounted for by the fact that the tests might be made on a machine such as one which is used in conjunction with a mercury column whereby a high result can be obtained if the load is put on too rapidly, whereas, for properly conducting a professional test, the test load should be put on at a low speed. In all tests a sudden load will give a much higher result than a gradually increasing load.

Effect of "Flour" in Concrete.—The question has often arisen as to the effect of very fine sand or so-called sand on concrete, and we have had two series of tests carried out on materials of this nature.

Table 3 shows the result of the grading of sand. Table 4 shows the test on cement used for the specimen pieces.

Table 5 shows the result of tests to ascertain the resistance of thrusting stress on *standard* sand and cement.

Table 6 shows the result of tests on the briquettes containing the very fine materials.

TABLE IV.

Test on Cement used for Tensile Tests.

Briquettes, 1 in. by 1 in.

Neat Cement.	3 parts Standard Sand to 1 part Cement.
Lb.	Lb.
572	196
527	238
570	206
551	192
556	191
Mean of 5 tests 555·2 lb. per sq. in.	Mean 204·6 lb.

Age seven days.

Residue upon sieve 50 × 50, 0·0; upon sieve 76 × 76, 0·0; upon sieve 180 × 180, 10·5 per cent. Time of initial set (25 per cent. water), three hours; set hard, nine and a half hours. Pats remained sound in air and cold water, also in water kept at a temperature of 115-120 degrees Fahr. for 48 hours.

Le Chatelier test for soundness: Cement aerated for 24 hours, expansion 3·5 mm.; after seven days' aeration, expansion 1 mm.;

1.	86·56	Weight of cement per cu. ft. obtained by various methods of filling cubic measure.
2.	86·28	
3.	85·11	
4.	85·80	
5.	94·10	
6.	95·25	

TABLE V.

Result of Experiments to Ascertain the Resistance to Thrusting Stress of Cubes of Standard Sand and Cement.

Description.	Dimensions.	Base Area.	Crushed.	
			Total.	Per sq. in.
	In.	Sq. in.	Lb.	Lb.
From Bin No. 22	3·00 × 3·00 × 3·01	9·03	41,440	4,589
Neat cement	3·00 × 3·00 × 3·00	9·00	43,480	4,831
Age, 7 days	3·00 × 3·00 × 3·00	9·00	42,240	4,693
	3·00 × 3·00 × 3·01	9·03	41,320	4,576
	3·00 × 3·00 × 3·02	9·06	41,000	4,525
	Mean	9·02	41,896	4,643
3 parts standard sand	3·00 × 3·00 × 3·00	9·00	10,420	1,158
1 part cement (by weight)	3·00 × 3·00 × 3·00	9·00	11,080	1,231
Age, 7 days	3·00 × 3·00 × 3·01	9.03	12,080	1,338
	3·00 × 2·00 × 3·01	9·03	10,440	1,156
	3·00 × 3·00 × 3·01	9·03	11,400	1,262
	Mean	9·02	11,084	1,229

Not sufficient quantity of material of "sample sands."
Nos. 1 and 2 to make comparative experiments to ascertain the resistance to thrusting stress.

TABLE VI.

Test on Briquettes Cement and Sand Samples 1 and 2 for Tensile Strengths.

Briquettes, 1 in. by 1 in.

Sand Sample No. 1.	Sand Sample No. 2.
Lb. per sq. in.	Lb. per sq. in.
112	178
108	180
102	170
103	191
100	168
Mean 105 lb. per sq in.	Mean 177·4 lb. per sq. in.

Proportions: 3 sand, 1 cement (by weight). Age, 7 days.

It will be observed from these tests that the average strength of briquettes with standard sand and cement gave a mean of 204·6 lb. at seven days, whereas the sand (Sample No. 1) gave only 105 lb. per square inch, and sand (Sample No. 2) which contained a less amount of flour gives an average result of 177·4 lb. per square inch.

On Table 4 the extraordinarily different weights per cubic foot of cement is given. The same cement was used, but different methods of filling were adopted. It certainly confirms the necessity of most carefully considering whether the cement for all work should not be measured by weight and not by cubic measure.

Another test, made as shown on Table 7, shows that with standard sand we get 182·2 lb. at seven days and 234·6 lb. at 28 days; whereas, with the test on "flour" sand as described on the table, we only get 53·2 lb. at seven days and 104 lb. at 28 days.

TABLE VII.

RESULTS OF EXPERIMENTS TO ASCERTAIN THE TENSILE STRENGTH, ETC., OF CEMENT AND ROCK DUST, AND CEMENT AND SAND.

Twenty specimens moulded here, immersed in water and tested at the ages of 7 and 28 days. Briquettes, sectional area 1 sq. in.

Cement and Rock Dust (3 *parts Rock Dust to* 1 *part Cemen* , *by weight.*)					Cement and Sand. (3 *parts Standard Sand to* 1 *part Cement, by weight.*)			
Age, 7 days.		Age, 28 days.			Age, 7 days.		Age, 28 days.	
Test No. QQ.	lb.	Test No. QQ.	lb.		Test No. QQ.	lb.	Test No. QQ.	lb.
4586 ...	64	4591 ...	135		4603 ...	194	4606 ...	264
4588 ...	56	4590 ...	100	Received Oct. 8th.	4599 ...	184	4605 ...	259
4587 ...	50	4593 ...	100		4602 ...	183	4604 ...	243
4584 ...	48	4589 ...	97		4600 ...	177	4607 ...	208
4585 ...	48	4592 ...	88		4601 ...	173	4608 ...	199
Mean of 5 tests	53·2	Mean of 5 tests	104·0		Mean of 5 tests	182·2	Mean of 5 tests	234·6

The rock dust used was that which passed through the 30 × 30 sieve. The Standard sand was passed through 20 × 20 sieve, and retained upon 30 × 30 sieve.

The tensile strength of the cement used, in making the above briquettes, was 480·0 lb. per sq. in. at 7 days, and 583·0 lb. at 28 days.

Residue upon sieve 50 × 50, 0·0; upon sieve 76 × 76, 0·9 per cent.; and upon sieve 180 × 180, 15·9 per cent.

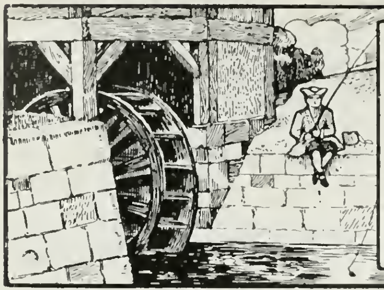
DAVID KIRKALDY & SON.

99 Southwark Street, London, S.E.
November 6th, 1908.

Specifications often provide for the work to be tested with one and a half times the load it has been designed to carry; a factor of safety of four is taken in the calculations to provide for inequalities of workmanship and materials, for allowance for fatigue of material under strain and for isolated accidental excess loading. Considering that with one and a half times the safe load the concrete in compression is working up to 900 lb. per square inch; adhesion 150 lb. per square inch; 200 lb. in shear, and steel varying up to 22,500 lb. per square inch in tension, the parts so tested by the application of such loads may be permanently injuriously affected.

If the materials are tested as suggested in this paper and professional supervision is given to the work, tests on the finished structure are not necessary; if it is desirable, then only the safe load should be applied, and if no undue deflection takes place no further loading can serve any useful purpose.

I have confined myself to giving you this evening results of tests which have been made from works I was associated with, and can vouch for all the information placed before you.



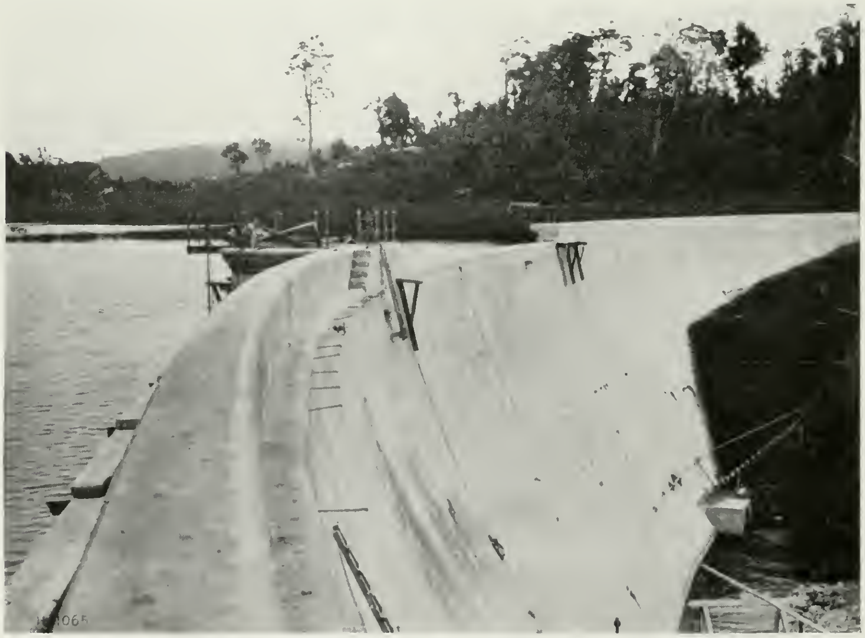
TWO LARGE CONCRETE DAMS IN AUSTRALASIA.

We have pleasure in giving below descriptions of two very large concrete dams, one for the water supply of Auckland, New Zealand, and the other at Coolgardie for the Western Australia water supply. It is gratifying to note the headway that this method of construction is making in our colonies.—ED.

CONCRETE IN THE CONSTRUCTION OF THE NEW WATER SUPPLY FOR AUCKLAND, NEW ZEALAND.

Owing to the increase in the population of the city of Auckland, it was found necessary to provide additional water for the city, and Mr. Hugh Munro Wilson, consulting engineer, of Auckland, to whom we are indebted for our illustrations, was entrusted with the work of surveying a source and carrying out a gravitation scheme. The following particulars of the new water supply appeared in our contemporary, *The Engineer*.

The site chosen was in the Waitakerei ranges, which lie about 17 miles due west from the city. After careful investigation, it was decided to tap the Waitakerei stream



View of Dam in Course of Construction.
Concrete Impounding Dam, Auckland Water Supply.

at a point immediately at the head of the celebrated falls, which have a sheer drop of 300 ft.

The works embraced in the scheme included, among others, 4 miles of bush road formation; $1\frac{3}{4}$ miles of graded track through the Foot Hills to take the 27 in. diameter locking bar pipe main from the break pressure reservoir in the open country; the formation of half a mile of mountain cutting, 10 ft. wide; the tunnel, 7 ft. by 7 ft., through the main range and half a mile long; the construction of a break pressure reservoir on the Foot Hills at the east side of the range and the construction of the concrete impounding dam, which is the largest by far in New Zealand.

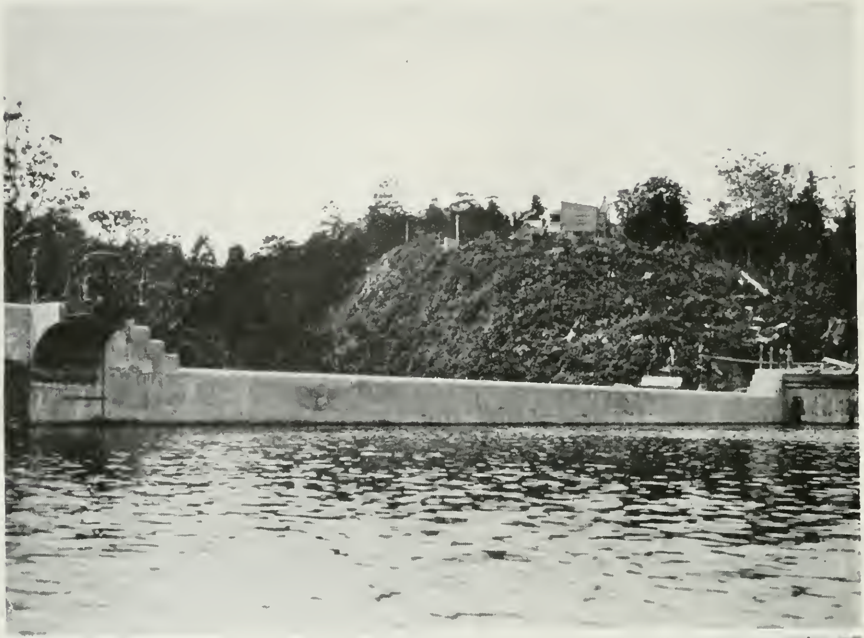
The break pressure reservoir is of concrete with a principal roof, and has a capacity of 1,000,000 gallons. Its top water level is at an elevation of 470·30 ft. The main tunnel offered no particular engineering difficulties. It went through solid ground composed of hard conglomerate and sandstone rock; it was practically dry throughout, and no timbering was required. The clearing of the impounding area was remarkable only for the enormous quantity of bush that had to be removed.

View of Valve Well in Course of Construction.
Concrete Impounding Dam, Auckland Water Supply.

The site chosen for the concrete impounding dam is admirable. It is so situated that it impounds every gallon of water discharged by no less than six different creeks or streams, draining an area of about 3,000 acres. In June, 1905, a start was made to clear the bush from the site in order to define the exact position of the dam. It was then found that it would have to be where two converging spurs were cut through by the stream, and to utilise these spurs for abutments a curved dam was decided upon, as a straight dam would have cut both spurs obliquely.

In designing the dam Mr. Munro Wilson decided to adhere to a gravity section, notwithstanding the additional strength gained by the curvature form, as a guard against earthquake shock. The curve is to a 382 ft. radius, and the total length at top water level is 533 ft. The lowest foundation is at level 602·41 ft. in the bed of the creek, and it rises gradually, and practically uniformly, to 642·00 ft. at the east end,

View of Spillway.



View of a Portion of the Dam.

Concrete Impounding Dam, Auckland Water Supply and 642·50 ft. at the west end. The top water level was originally fixed at 666·50 ft., but was afterwards raised to 668·50 ft. The foot of the parapet is 671·50 ft., and the latter is terraced at the ends to 673·50 ft., thus giving a total height of 71·09 ft. from the lowest foundation level to the top of the parapet. The upstream face batters $\frac{3}{8}$ in. to the foot, and the down-stream face 8 in. to the foot up to the 649·50 ft. level, from which point the up-stream face graduates to a vertical at the 668·50 ft. level, and the down-stream side curves at a tangent and gives a crest top width of 3·50 ft. The parapets are 3 ft. high and 2 ft. thick, and an overflow weir, 100 ft. long, has been left in the centre.

The water is drawn off by a vertical 20-in. cast-iron pipe column standing in a semi-circular chamber on the up-stream side; within this chamber are assembled all the valves, which draw the water at three different levels. The approximate content of the storage reservoir is 290,000,000 gallons, and the submerged area is approximately 50 acres.

The total quantity of excavation in the foundations was, approximately, 7,000 cubic yards. The concrete was of two kinds, numbered 1 and 2. The latter was used only on the up-stream face, where it was 6 in. in thickness. No. 1 was used throughout the remainder of the structure. No. 1 concrete was composed of 18 cubic ft. of $1\frac{1}{2}$ in. metal, 9 cubic ft. of $\frac{3}{4}$ in. metal, 11 cubic ft. of sand, $4\frac{1}{2}$ cubic ft. of cement. No. 2 was composed of 18 cubic ft. of $\frac{3}{4}$ in. " shivers," 11 cubic ft. of sand, $4\frac{1}{2}$ cubic ft. of cement.

All concrete was mixed by hand in cubic yard batches, and in the following manner. The whole of the materials were gauged in cubical boxes. The sand and cement were first mixed together by being turned twice dry. The metal was spread 6 in. thick, the " shivers " were equally spread over it, and the whole was covered by the combined sand and cement. The mass was then turned twice dry. Water as required was added during the third and fourth turns, and the whole then received two final turns; six turns in all. Owing to stone " plums " being used, the water was to a certain extent forced out of the concrete again; therefore the quantity of water used per batch continually varied, as some batches had to act as an absorbent to prevent loss of matrix; for this reason Mr. Munro Wilson preferred hand to machine mixing, as the constantly varying conditions were more under control.

Before laying the concrete the portion of foundation to be covered was washed by nozzle under 45 lb. pressure per sq. in., and made absolutely clean. Upon the surface so obtained a thick coat of pure cement grout was laid, and heavily crushed into the rock surface, and upon this was laid a $\frac{1}{2}$-in. coat of 2 to 1 cement mortar, and the concrete was placed immediately upon that.

The No. 1 and No. 2 concrete mixtures were placed in position at the same time, and well rammed together. Stone " plums," with a minimum size of 1 cubic ft. in every direction, were inserted, 12 in. back from the up-stream face above all foundations, and 9 in. back from the down-stream face. The stones were washed under pressure before using, and were well ground down into the concrete, which was used very wet and practically flowed round the stones till they were covered to the requisite depth; upon which another layer of stones were bedded. Spading and ramming were kept continuous. No concrete was tipped from barrows into place, but was placed by shovel from a small bunker at the face.

The whole work was built in courses 3 ft. high, accurately levelled and kept continuous till the course was completed from end to end. All the concrete, when in place, was covered with bags kept constantly wet for four days, and after the bags were removed the set concrete was kept constantly wet by hosing till the succeeding course covered it. All concrete surfaces were washed under the usual pressure, and well grouted. This system of building was specified throughout, and was continued up to the 654·50 ft. level. The contractor then requested to be allowed to substitute massed concrete owing to the want of room. This was assented to, and the work was completed with the No. 1 concrete in mass laid in 6-in. layers, and tipped from the barrow.

There were 160 ft. of the east end, and 190 ft. of the west end of the dam in trench, which receded to 40 ft. at each end at top of dam. The up-stream side of the trench was dressed neatly and vertical, and against this surface, after being well washed and covered with a thick coat of neat cement grout, the concrete was laid and well rammed into the surface, no timbering being used. At the ends, and also 30 ft. from the ends, chambers were cut into the bank 4 ft. by 3 ft. These were prepared, as was the rest of the face, and filled with concrete. They were cut out from the bottom upward for each 3 ft. course, the continuous line of trench and concrete being thus broken. These chamber locks started at the foundation and continued to 2 ft. above top water level. The top of the dam was finished off with 4-in. and 1½-in. nozings, and all horizontal surfaces were covered with ¾ in. 2 to 1 cement rendering. This was the only rendering on the whole work, the remaining concrete from bottom to top being left as stripped from the moulding boards.

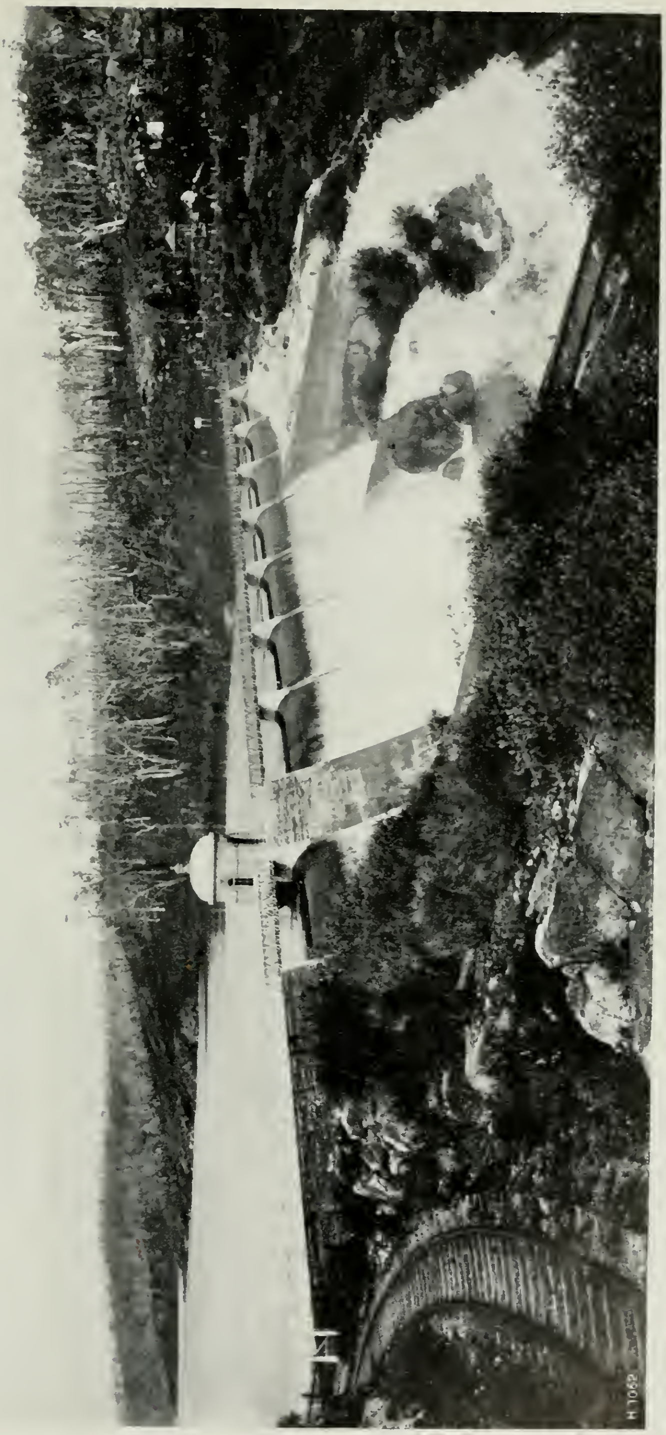
The method of construction, as detailed above, has, we understand, proved perfectly satisfactory. The dam is stated to be absolutely water-tight, there not being a sign of moisture on the down-stream face, nor over the inside of the valve chamber, the wall of which, though only of an average thickness of 3 ft. 9 in., being absolutely dry with 40 ft. of water against it at the present time.

A CONCRETE DAM AT COOLGARDIE FOR THE WESTERN AUSTRALIA WATER SUPPLY.

Having a daily supply of 5,600,000 gallons of water, with a total length of 351½ miles of conduit from the supply reservoir, Coolgardie, the great goldfields city of West Australia, has completed a very comprehensive scheme that has many fine engineering features, at a capital cost of £2,500,000, the period of construction being about five years. The weir was built to meet a maximum pressure on any portion of the wall that should not exceed eight tons per square foot, and that the centres of pressure should be well within the middle third, both with the reservoir empty and when 5 ft. of water was flowing over the crest.

Preliminary Works.—The reservoir and the first pumping station are situated at the bottom of a deep valley some miles from the nearest railway, and as all material, except stone, for the concrete of which the weir was built had to be brought from a distance, the first work put in hand was the construction of a tram-line to the standard railway gauge of the State, starting from Mundaring station on the existing line of railway. The next question was the provision, at a comparatively waterless spot, of a permanent supply of water fit for the use of the many men to be employed, as well as for the works. The requirements were met by constructing, in the bed of the future reservoir and about nine chains above the weir wall, a temporary concrete dam, impounding about 20 million gallons, and by forming, from the by-wash with which this small reservoir was provided, a channel capable of carrying away 100,000,000 gallons a day.

Foundations.—Generally speaking, the country at the reservoir site is very rocky, consisting largely of undecomposed granite, traversed by intrusive basaltic dykes whose direction is mostly at right angles to the course of the river. At the site of the weir, however, the granite showed out particularly clearly, and the few trial shafts, put down where the rock was shattered, reached solid at no great depth, the deepest of the shafts being only 20 ft. deep from the ground-surface. On opening up the foundations, however, it was discovered that the rock was nothing like as solid as



Immense Dam Built of Concrete at Coolgardie, Western Australia.

surface indications and trial pits promised; for on the right bank a large portion of what at first appeared to be bed-rock was found to consist of an immense boulder with a large cavity below it; and under the bed of the river the granite was very badly fissured over the full width of the foundations. It was not possible to vary the site, as the disruption was found to extend both up and down stream for a considerable distance, and there was no alternative but to follow the fissure down, which was done until a depth of 90 ft. below river-bed was reached. It having been seen that the fissure had a northern underlay, a vertical boring was now made on the north bank of the river and was continued for a further depth of 52 ft. The bore was then filled with water, and the material in the fault was subjected to a hydrostatic pressure equivalent to a head of 690 ft., which was maintained for four hours, during which time the foot and hanging walls of the fissure and the line of the fissure of the excavations were all carefully examined; but no sign of moisture could be detected. It was concluded that the material in the fissure at the depth which the excavations had then attained was impervious to water and that it would therefore be safe to erect the weir thereon.

Where the wall would be highest—that is, where the fissure occurred in the foundations—the excavations were carried down about 15 ft. from the building-line in a vertical direction on the up-stream face; but as one of the basaltic dykes crosses the valley a short distance away on the down-stream side, it was considered necessary to remove the whole of the material between the hanging wall of the dyke and what would otherwise have been the toe of the weir. The concrete filling of foundations was carried up to bed level on the up-stream face, but on the lower side the mass filling was stopped 18 ft. below bed-level and the wall proper was begun to the designed section. The granite beds, or floors, were deeply chased in longitudinal rows about 6 ft. wide and 3 ft. deep, and the toe of the wall-batter, where it met the granite floor, was channelled the whole length to key the concrete in.

The great inequality in the depth of the foundations, and their apparent doubtfulness for a work of this magnitude, have not appreciably affected the weir; for although very fine vertical lines, such as invariably occur in the concrete lining of service reservoirs in hot countries, have made their appearance here, they have not extended, and any slight sweats have taken up.

The reservoir is provided with two valve-towers constructed of concrete. The inner tower (situated on the reservoir side) of the weir was built into, and concurrently with, the main wall, being approximately semi-circular in section. The outer valve-tower is rectangular in section, and is situated 175 ft. down-stream from the centre of the weir-wall, being connected therewith by a viaduct which carries the outlet and scour pipes, all solidly bedded in concrete, as far out as the outer valve-house.

Provision is made for drawing water from the reservoir at three different levels, namely, at 25½ ft., 53 ft. and 80 ft. below full-supply level, by means of 24-inch cast iron bell-mounted pipes, passing through the valve-tower wall into a cast-iron stand-post. Each draw-off inlet is provided with a stop-valve placed in the valve-chamber, from which valve rods are carried up to bevel-geared headstocks, all placed on the upper valve-tower floor, which is 1 ft. 9 in. above maximum flood-water level of the reservoir.

A concrete lined spill-water basin, about 150 ft. long by 100 ft. wide, is constructed in the river-bed at the toe of the wall, with a depth of water of about 10 ft. The water is confined by a mound across the river-bed, constructed of rubble, faced with concrete.

The Cement.—In the construction of the weir wall 76,418 casks of Portland cement were used, and a further 1,090 in the spillwater basin and other subsidiary works, making a total of 77,508 casks. A cement which did not demand much slaking before use was especially necessary in connection with the smaller scattered works of the scheme, distributed as they were over 350 miles. In these smaller works the cement, having passed the necessary tests, was used direct from the casks, because to slake and then repack it would have entailed considerable expenditure; but at the weir provision was made for slaking fully all cement requiring it.

The tests carried out on the cement were of an exhaustive character. Situated as the works were at such a distance from the source of supply this was essential. In all ninety-two complete analyses were made and, taken altogether, the cement received was very satisfactory. The tests of tensile strength ranged from three-day and seven-day hot and cold bath tests to twenty-eight-day cold bath tests, a reserve of briquettes being often retained from the various batches for long-date tests.

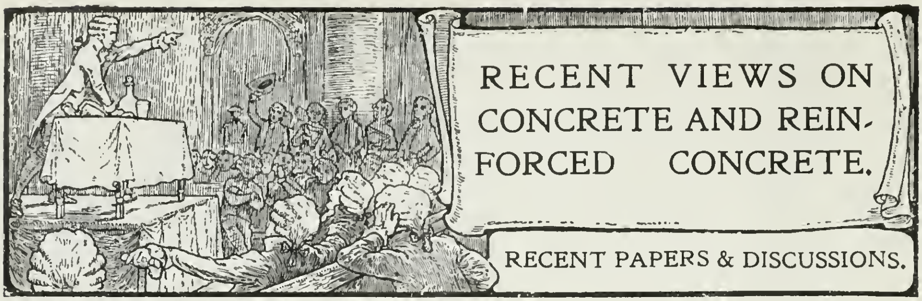
Below the cement shed, on the same level as the quarry road, the stone-crushing plant was erected. In the same shed, but below the crusher, was situated the concrete mixer, portable and self-contained, of the rotating barrel type, mechanically fed with cement, stone, and sand in the proper proportions, and capable of mixing 20 cubic yards of concrete per hour.

Except about 1,000 cubic yards of sharp, coarse-grained sand obtained from the river about one mile below the weir site, the whole of the sand was brought from either Lion Mill or Bayswater, distant eight miles and twenty-two miles, respectively, by railway. That from the former was of quartz, and very fine grained, yielding even and good results in the testing-room.

About 30,000 cubic yards of spalls, for crushing to concrete size, were selected from the material obtained in the excavation of the foundations. For the plums and the balance of the spalls required a quarry was opened on the north bank of the stream, below the weir, and about 70 ft. above river bed.

The Application of the Concrete.—The weir and all accessories were built of concrete, but in the former large rough granite blocks, just as quarried, were introduced into the concrete. It was originally intended to build the wall with 50 per cent. of these large blocks; but without proper plant, which was not available, handling would have been very expensive. The concrete consisted of five parts by measurement of granite crushed to 2½-inch gauge, two parts of cleaned sand and one part of Portland cement. So long as the wall remained below the level of the mixing-house the mixture was conveyed to the work on an endless conveyor, working in a trough, with travelling boards secured by ropes and spaced 2 ft. apart, thus ensuring that the heavier aggregate was not separated from the matrix on the way. Later the concrete was conveyed on a trolley-line in skips to a large derrick crane, which lifted the skips on to temporary tram-lines on the growing wall; they were then pushed by hand to a travelling steam-crane which lifted each skip in a bridle, overturned, emptied and returned it to its carriage. The concrete was spread and rammed by hand, the various layers being broken up, so far as the width of the wall would allow, in order to break bond in both beds and joints; and in addition the large rough blocks, up to 2 cubic yards in volume, were deposited and thoroughly bedded and grouted in order to key the bedding planes together.

For the first 10 ft. the batter was lined and the concrete retained by rubble masonry, but above this level wooden framing was substituted. This framing was of Oregon pine, and consisted of uprights 9 in. by 3 in. and 15 ft. long, cut to the sweep of the wall section, spaced 3 ft. apart, and closely lined on the wall face with 12-in. by ½-in. Oregon boards. Rendering of the face was not desired or found necessary, as great care was taken when depositing the wet concrete in contact with the moulding boards to keep all stone well back with straight spades, and a good finished face resulted on stripping.


RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.
RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE PAINT AND VARNISH SOCIETY.

THE PROTECTION AND FINISHING OF CONCRETE AND CEMENT SURFACES.

Paper by MR. ROBERT CATHCART.

The following is an abstract of a paper read on March 21st last before the Paint and Varnish Society by Mr. Cathcart, who is well known in connection with concrete work in America.

THE protection and treatment of concrete surfaces in recent years, owing to the increasing usage of concrete and cement in modern building construction, have been problems necessitating very careful investigation and research by the engineer, architect, surveyor and builder. Many ideas for the accomplishment of the above have been suggested and tried. But the great difficulty that immediately confronts the average investigator is the applicability of the protective or finishing materials to that of concrete.

Appreciation of the connection which exists between the physical elements of concrete and those of other building materials, such as natural stone, brick, terra-cotta, wood, etc., and their relative differences to weathering and wearing, has to a large degree limited exterior concrete surfaces.

From various opinions of architects, engineers, and builders, it would seem that a proper method for the protection and finishing of external concrete surfaces would undoubtedly promote the extension and use of monolithic concrete structures, which under existing conditions have to be faced with building stone, terra-cotta, or brick. However, Portland cement concrete is a fire-resisting and economical building material, and its great utility in all classes of construction has placed it in a position beyond that of others, and its needs for protection and finishing are imperative. The great obstacle to the decoration and finishing of concrete has been, in the main, first, that no particular attention was paid to the selection of the aggregates in the mix, or their influence upon the resulting surface; second, the methods of mixing and placing the aggregate and forming.

Concrete, therefore, is subject to many conditions which make the methods and modes of protection and finishing a difficult problem—for instance, discoloration and variability of colour, which are due probably to the bleaching out of the lime compounds and their disposition upon the surface in the form of " efflorescence "; or, in cases of excess water being used in the mixture, the surface in such places would give a " laitence " face, which is a thin impermeable formation caused by the lighter particles of the cement floating to the surface, or roughness due to the irregularity of the " form " work; carelessness in the preparation of the form to receive the concrete, or too long a period elapsing after the preparation of the form to receive the concrete, deposits of foreign matter, such as sawdust and hardened mortar, the choking of the mix in the forms, or insufficient tamping.

The density and porosity of a concrete surface is largely governed by the amount of the gauging water, its mechanical mixture, and the mechanical method of placing. The main object in procuring density in concrete is for water-tightness, the protection of the metal reinforcement from corrosion, and from loss of its adhesion with the structure. We have therefore three surfaces to consider in the protection and finishing of concrete :—

(1) ***Dense Surfaces.***—These are generally obtained by the careful selection of the aggregates, their method of mixing, and placing under normal temperatures.

(2) ***Porous Surfaces.***—These may be caused by the leanness of the mortar, or a fat mixture concreted dry, or the loss of the mixing water by absorption through the forming or insufficient tamping, the addition of bad mixing water, adulterated additions to the cement, concreting in heat or cold. This concrete is particularly sensitive to temperature changes, and the metal may be corroded by the presence of moisture entering through the porous surface.

(3) ***Mortar-face Surface.***—This is generally produced by the excessive use of water to obtain impermeable surfaces for reservoirs and other heavy construction work, or the excessive damping or tunning of the aggregate to the back of the form. This concrete, when applied to the forming, is generally what is termed excess moisture. It has one very great objectionable feature regarding the surface protection and decoration in that it "crocodiles" and spalls owing to shrinkage, due to the difference of the coefficient expansion or contraction of the outer or inner surfaces. This is a surface that has caused a great deal of criticism on external concrete surfaces because of the light hairline cracks—which we call in the paint world "alligating"—and the small areas of surface which spall or scale, giving the average layman an idea that a concrete external surface is not one that would compare with the average natural or artificial facing materials.

"The external influences of the atmosphere and atmospheric gases upon surface elements of concrete demand a treatment for its preservation."

Surface elements show their destruction in various forms of disintegration :—

Cleavage and Parting.—The swelling and contraction of the crust against the grain bond and the binding substance by the alternate action of heat and cold. During the calcination of the materials the chemically combined water is driven off, and during this process a shrinkage due to the withdrawal of this moisture in concrete is a very important factor. For instance, in a 1 : 2 : 6 concrete the coefficience of shrinkage during the calcination is equal to about ¾ in. per 100 ft., while a temperature of 121 deg. Fahr. would be necessary to cause the same contraction as results from the loss of moisture. The coefficient changes with the various richer or weaker mixes. The additions of hydrated lime and lime compounds are sometimes added to mixes to equalise this condition, but, when added in excess, generally swell and disintegrate the surface. They have not to any degree of satisfaction fulfilled the purpose.

Fracture.—The loss of adhesion by the softening action of water against the grain bond and binding substances. A surface may appear perfectly dry, yet percolation may be actually taking place. Water can pass through construction so slowly that the evaporation from the outside proceeds more rapidly than the water can reach the surface.

Permeability.—The saturation of the porous strata by capillarity, gravitation, or immersion under water, or the gaseous products of combustion in the atmosphere. It would seem that the filling of the superficial voids upon the surface with water or vapours would have a tendency to fill those voids with a crystallisation of the salts in the pores, and this would be accompanied by swelling, causing disintegration. For protection against such entrance, a strictly waterproof repellant would seem necessary.

Hardness.—The inability of the binding substance, cement, to resist friction, causing powdering or "sanding up." This is applicable to both weathering and wearing surfaces. "Sanding up" on concrete may be due to a great many mechanical causes—soft sand, loam or other foreign matter in the sand, poor mixing, excess sand over cement, loss of time between mixing and placing, mixtures too wet or too dry, the use of dryers, the lower layers of concrete not being able to take care of the excess moisture in the mix. This surface cannot be trowelled owing to its moist condition, taking its initial set before it can be finished, thus reducing its strength.

The problem of available means and resources for the treatment of concrete surfaces has generally been governed by the materials that were depended upon for supply and general custom. Therefore cements and limes, made into plasters for rendering, have probably been the most available sources of supplying the need for their necessary protection and treatment.

Concrete exterior finish for large, massive work, such as bridges, abutments, factories and general building exteriors, has been subject to treatments which may be termed mechanical. They are executed either with a mechanical or hand device by skilled or unskilled labour, and are known as picked surfaces, scraped surfaces, rubbed surfaces, bush-hammered surfaces, tooled surfaces, sand-blasted surfaces, etc. All of these mechanical treatments have been worked towards a betterment of surface appearance; but they do not tend to give any protection, and, being for effect only, they have no weather-resisting properties.

The proposed specification for scrubbed concrete surface, issued by the National Association of Cement Users, is to scrub the original surface immediately on removal of forms until the surface film is removed and the aggregate exposed to a uniform degree. It also includes the filling of void spaces and the removing of hard spots with a 1 to 10 solution of muriatic acid and water. This method should give a betterment towards the surface conditions for the protection and finishing of the surface. This method can only be done while the concrete is in a fragile state, which may mean the day after placing or later, depending upon the weather and setting of the cement. However, the increased cost for the removal of the forming, and the danger pertaining thereto on various classes of construction, might tend to increase the danger of disturbing the aggregate and reinforcement during the hardening process, although there are quite a number of surfaces that can be treated in such a manner.

There is another method—namely, chemical—closely resembling that of the scrubbed surfaces, which removes the original surface by the use of a solution of commercial muriatic acid while the surface is still green, after the forms have been removed. This method of treatment has been patented in the United States. Its method is to expose a selected aggregate, which is placed towards the face of the forming, specially selected for its lustre and general appearance. For small members cast in units this method is making some considerable headway; but in monolithic construction the difficulty of obtaining uniformity of the aggregate giving texture, and the cost of removing the outer surface, with the objectionable feature that the muriatic acid may stain or discolour the aggregate, and the excess in cost, do not warrant this method of treatment. From experience of the mechanical, chemical and rendering methods, they seem to be actually condemned where there is the slightest presence of moisture.

The paint chemist has put forth a considerable amount of energy towards the production of materials for the protection and finishing of concrete surfaces. He has confined his investigation generally along two lines:—

First, to obtain a material, coating or size that would neutralise the free lime upon the surface; second, a superficial filling of the pores upon the surface to stop suction. Much experience has been derived from the first line of investigation, and the principal exponent of this method has been Mr. McNichol, of Washington, D.C., who used a solution of 8 lb. of zinc sulphate to 1 gallon of water, for the neutralisation of the free lime upon the surface. Other methods have been weak solutions of hydrochloric acid, applied to the surface, to neutralise the free lime. The second method of filling the superficial voidage has been to use substances soluble in water or naphtha applied to the surface, such as soap, fluxes of wax or grease, soap and alum.

The first and second lines of investigation developed what may be readily termed shallow superficial treatment for concrete surfaces. It was one that was much in request by architects, engineers, surveyors, and builders during the earlier stages of concrete. Its apparent advantages were that it filled the superficial voidage at the surface, and gave the surface a protection against moisture and discoloration. The failure of the superficial method which consists of the introduction of a solid substance in an aqueous or volatile medium is that the volume of the solid, as introduced into the voidage, as compared with that of the medium, is so small that, after evaporation takes place, so small a quantity of solid is left that, although itself insoluble in water, it is easily washed out with heavy rain or seepage. Many patented processes of paraffin wax in solution for the treatment of concrete, brick and stone surfaces, and applied by heat, are carried on by waterproofing and stone-restoring contractors.

The protection and finishing of concrete and cement surfaces now resolves itself into the consideration of two factors :—

First, the necessity of maintenance against disintegration; second, their finishing and decoration. Before passing from external weathering surfaces let us consider a concrete wearing surface.

Floors. — Concrete floors, when laid and finished under the strictest specifications for density, texture and uniformity, are sometimes subject to external injury, temperature changes and varying conditions, owing to the time, place, and method of completion. The outer character of integral and bonded wearing surfaces and their surface elements are much the same as those described for external surfaces. After mixing and trowelling, very little free water should appear upon the surface, even after thoroughly spreading, as an excess of water in the mixing is likely to crack the finish, due to shrinkage.

A mistake that is sometimes made on concrete floors is that they are sprinkled when hot, during their setting period, and often they are sprinkled while the mortar is too soft to resist the washing action of the water. The dusting of top material is quite a detrimental factor to the wearing surface and produces a surface element which indents, spalls and scales, and gives other bad defects to the floor, although slight freezing would give exactly the same effects, causing disintegration. Smoke gases from open braziers which are often placed in the building to increase the temperature have a tendency to retard the setting, on account of their carbon dioxide exerting a harmful effect on hydraulic cements setting at a temperature of about 42°. This action may be only a superficial one, but will cause a dusting up of the floor.

The maintenance and protection of a concrete floor is an important factor to their service. The determination of the cement and the sand in concrete floors to lose their elements of adhesion as a wearing surface is a well known fact. The frail constituents of the binding substance are removed by the softening action of water, oil, grease, acids, etc. The harder constituents of the grain bond are loosened by their loss of adhesion and worn down by friction.

Waterproofing against a hydrostatic pressure is an engineering problem of much moment, and generally governed by its individual character. Therefore it seems unnecessary to treat this subject here at any length, because of its purely engineering character.

The subject of finishing and protection may be divided into three distinct methods —integral, superficial, and physical. The integral method of finishing concrete is one that may carry with it a colouring material, combined with the waterproofing material for producing a surface texture and finish, applied either by a rendering or integral with the construction at the time of casting or concreting.

The superficial method of finishing a concrete surface may be that of scrubbing the surface with water or acid, and then filling the superficial voidage with a cement wash. This wash is sometimes coloured to gain uniformity, or used in the natural colour of cement. The objectionable features to this method of finishing an external concrete surface are: that the amount of water used, in comparison with the solid cement, is so large that the surface is never a homogeneous compact mass of solid material. The amount of voidage under this surface caused by the entering of the water and the difference in expansion and contraction of this wash or grout to the inner surfaces causes the cracking, scaling and spalling of this light crust. This is the surface that has caused many concrete decorative failures, because of its lack of binding properties.

The Physical Method.— This is a coating of materials specially adapted for preventing contact between the atmospheric elements and the construction. This method should carry with it two features in one operation. They should be damp-proofing and decorative.

In conclusion, a line of investigation to follow for the testing of materials for their applicability for the protection and finishing of concrete surfaces may be outlined as follows :—

It would seem necessary in the light of investigation that the use of acids or acid re-agents, used as primers or vehicles for concrete weathering or wearing surfaces, would not be advisable for the reason that they would leave a very unstaple bond to apply any succeeding treatment.

The protective properties and decorative effect of a treatment and its resistance to moisture and decomposition by free lime, water, acids, oils, and gases seems to be the necessary qualifications for an article manufactured for this purpose—that is, the protection and finishing of concrete surfaces.

DISCUSSION.

Mr. J. G. McIntosh said the author had condemned the acid treatment as a whole, but there were acids and acids, and he would draw attention to linoleic acid, which would combine with free lime to form an insoluble lime soap, and he wished to know whether a dilute solution of that acid, either in white spirit, or, better still, in methylated spirit, applied to concrete would be of any effect. It was hardly possible to apply any paint to a very wet surface, but Mr. Toch assured him it was possible to apply a certain paint, the formula of which he did not divulge, to green concrete. Then he should like to know whether the present grey colour of concrete was necessary. As far as he understood, the clay ingredient was too frequently Thames mud, whose colour could not very well be altered, and the lime was made from chalk, which when burnt was a pease-pudding yellow, slightly whiter when it was slacked. There were occasions where cost was no object, when a white, higher-class concrete might be advantageous, when china clay might be used as one of the ingredients and a high-class white lime might be another ingredient of the Portland cement for making such a concrete, the sand used being silver sand. Mr. McIntosh also asked what coating Mr. Cathcart would recommend to protect the floor of a sugar house, seeing that lime is soluble in sugar liquors. Would a solution of sugar do to dissolve the lime efflorescence on the concrete instead of hydrochloric acid? For one thing, it would not form a deliquescent compound with the lime, like hydrochloric and acetic acids. He would also like to know whether it was the paint used on the concrete buildings of the town of Baltimore that caused the conflagration that occurred there some two years ago to spread with such phenomenal rapidity, and if it was the paint that caused this, whether it was the indoor or the outdoor paintwork, or both?

Dr. Blackler said that undoubtedly the mechanical treatment necessary for the production of good, sound concrete was not always duly considered. The various ingredients required careful selection, and as the process was one that was carried out in England by unskilled labour unless proper supervision was exercised, so that the constituents of the concrete were thoroughly incorporated, failures were very liable to occur. Facing the concrete surface with cements, Mr. Cathcart said, had proved unsatisfactory because of the "alligatoring" that took place, but paint chemists were much more interested in the processes of applying paint to the surface of the concrete. For concrete flooring the use of sodium silicate had been suggested, and it was stated that it hardened the surface, and to a great extent acted as a waterproofing material. The protection and decoration of concrete might be divided into two parts. The free lime formed in the concrete during the process of setting acted upon the linseed oil in a paint and the oil film was destroyed; so, firstly, it was necessary to eliminate this action by the use of special washes or undercoats; and, this done, the generally adopted methods of decoration and protection applied. If green concrete were allowed time to mature, carbon dioxide was absorbed, and in twelve or sixteen months the surface might be coated with a linseed oil paint with good results; but if it was desired, as is mostly the case, to paint a concrete structure soon after it was erected, it was necessary to destroy the lime, which could be done in several ways. In the first place it could be effected by the production of an insoluble lime compound without the production of another insoluble material. For that purpose oxalate of soda had been used, a material that gave an insoluble calcium oxalate and a soluble soda salt. It was stated that that method gave satisfactory results, but he presumed that very soon the soda would act upon the paint film. Acid phosphate of lime was a material which when applied combined with the lime of the surface, giving an insoluble compound. Resin, mixed with stearic acid, combined with the lime, and the stearic acid acted as a waterproofing material, but had not given satisfactory results. Soap emulsions had been also used, but when concrete surfaces were treated in that way and afterwards painted the linseed oil film disintegrated in about five or six months. He thought caseine paints might be used with great effect on concrete structures. Suitable caseine solutions applied to a concrete surface, and when dry treated with formaldehyde, gave a coating which was absolutely impermeable to water, and formed a surface on which paint could be laid. Another method was the production of innocuous lime compounds with the simultaneous formation of other insoluble bodies. The McNicol process, whereby solutions of zinc sulphate were employed, was one that had been widely used in the United States and gave very good results, superimposed paint films having lasted for two years without anything detrimental happening. Solutions of alumina sulphate and magnesium sulphate had also been applied with equally good results. With regard to coating containing pigments, mixed pigments containing free silica had been applied over concrete coatings and had not proved so satisfactory. Zinc sulphates had been formed into an emulsion, also with unsatisfactory results. He thought that the copals when made up with a small quantity of oil and diluted with turpentine would form a good waterproofing coat, and one extremely suitable for the treatment of concrete.

Mr. Noel Heaton said he thought that the point they were more particularly interested in was how to render concrete suitable to take decoration, and that was quite a different problem from the one Mr. Cathcart had dealt with, the protection of concrete itself. It seemed to him that the protection of concrete from disintegration must be something more or less of a makeshift, shirking the difficulty. The real way to get over the difficulty was to study the problem of improving the manufacture of the concrete itself, because any treatment such as impregnation with organic materials, etc., would be really less durable than a properly made concrete. He himself had always looked upon a good concrete as a material of great durability, certainly more durable than many building stones. Linseed oil and linoleic acid were not permanent materials. He had often been called in by architects to advise them how to deal with the decay of stonework in old churches, and had frankly said that whatever was done was more or less of a palliative, that the proper method was to select the stone with care so that it would not decay. It was the same with concrete.

Mr. Matt. Garbutt said up to the present the architectural finish of concrete had been neglected The new Post Office buildings in King Edward Street were fronted by another material. Some few months ago Professor Beresford Pite, at the Concrete Institute, seemed inclined to endorse the method of the old Romans who veneered the structure with marble mosaic or slabs of marble, in fact to beg the question and not really finish the concrete itself at all. In mass concrete as opposed to ferro-concrete it was quite possible to get a finish that would be satisfactory to the eye. He thought Mr. Cathcart, when he referred to muriatic acid treatment, really meant surface pickling, to expose the aggregate and thus obtain a roughened texture, which was rather pleasant.

Mr. Cathcart, in reply, said if it was admitted that reinforced concrete had porosity, it must be remembered that building stone with porosity had tensile strength, so had reinforced concrete tensile strength on account of its porosity. With regard to the fire in Baltimore, it would not have been so disastrous as it was if the buildings had not been faced with natural building stone, terra-cotta, and artificial building materials that calcined under the fire. With regard to the manufacture of cement, he believed the Engineers' and Architects' Societies had issued a standard specification for cement which was worked to by all Portland cement manufacturers very closely, therefore the cements used in Great Britain were mostly pure and unadulterated. But cement was not the only factor; there was the sand and the aggregate. Sand had a certain amount of influence on the surface, and the variance of sands in colour and shape had, of course, great influence on the porosity of a cement surface. Cinders were inert when they were properly washed and cleansed, but they were generally used for floor construction or as an insulating base for roofs. When speaking of acid treatments for exterior surfaces, he meant that the acid was used to remove the small film of mortar on the face so as to give a surface texture. The aggregate might be composed of white silica sand and small broken granite, and sometimes felspar and carborundum, with occasionally broken brick or tiles or marble or gravel. In connection with the aggregates, much could be planned ahead for the treatment of the surface. The principal objection to exterior concrete surfaces was not that concrete was not a good surface material, but that the art of making the forms had not progressed far enough. In the United States there were many systems of forming, some of which gave a surface texture of value, and as the methods of forming progressed the surface texture would undoubtedly improve. The work needed skilled labour, and when that was recognised there would not be so many failures with regard to damp walls.

With regard to the paint for concrete, any specification he might give for a paint for a concrete surface might not apply to all concrete surfaces. A complete specification for a paint for a concrete or cement surface required a thorough investigation of the surface itself. The problem of a specification for paint naturally had to include the time at which the paint had to be applied to the surface and the condition of the surface. Generally speaking, a paint applied to concrete would cover probably about one-third less than lead paint on wood. The second coating would cover a slightly greater surface.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

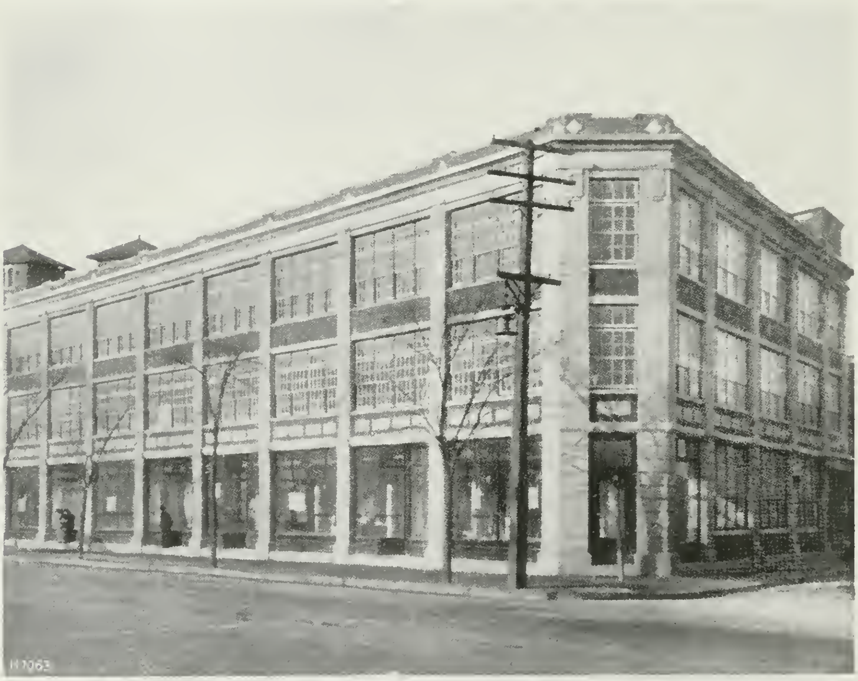
REINFORCED CONCRETE FACTORY AT CAMBRIDGE, MASS.

Recently at Cambridge, Mass., U.S.A., there has been completed a triangular shaped reinforced concrete factory owned by the Cambridge Factory Trust. It will be noted from the accompanying illustration that the exterior of the building has a very dignified and refined appearance, while the interior is arranged for maximum efficiency. The design was made in accordance with plans and specifications of Monks and Johnson, architects and engineers, of Boston, and the type of construction was adopted on account of its fitness for close quarters and other reasons.

The column and flat slab system of construction was adopted, for it was found as cheap as the beam and girder type, besides offering the additional advantages of simpler forms, lower cost of automatic sprinkler installation, and better overhead shafting facilities. The forms for a beam and girder system would have been very complicated owing to the triangular shape of the building.

The factory is three stories high with basement, with all floors and the frame of reinforced concrete, while the exterior between the pilasters is veneered with 4-inch red sand-struck brick. The longer side of the building is about 151 ft., while the shorter sides are 87 ft. and 132 ft.

The general impression that reinforced concrete structures have a displeasing effect



View of Completed Building.
Reinforced Concrete Factory at Cambridge, Mass.

is being diminished by such structures as this Cambridge cigar factory. The lines of the building are exceptionally well kept, the height seeming more than it really is, due to the continuation of the pilaster lines to the top. This is effected by means of corbels and small marble panels at the top. The entire width between the pilasters is taken up by the large window area beneath, which is 4-inch brick veneering with concrete panels in the centre and below each of the four respective windows.

The entire cornice is of concrete, while a concrete coping rests on the 8-inch thick brick wall running around the top of the building. The artistic effect is excellent, and the building is a very creditable addition to the community.

The interior arrangement of the building has been planned for maximum efficiency. All the floors are of 7-inch concrete, the first floor having a terrazzo finish. There are three rows of columns on each floor running parallel to the long side of the building with 18-ft. bays each way. The window area is about 75 per cent. of the wall area, this being secured with windows 7 ft. 6 in. high running up to the floor slab.

The columns, rectangular in cross section, vary from 20 in. to 30 in. in the basement and first floor to 16 in. by 30 in. on the second and third floors. The basement columns are reinforced by 12 1-in. diameter rods with $\frac{1}{4}$-in. hoops placed 12 in. on centres. The reinforcement of the first, second, and third floor columns consists of $\frac{3}{4}$-in. rods, eight being in the first-floor columns and this number decreasing by two for each successive floor. Hoops $\frac{1}{4}$ in. in diameter are also used, spaced similarly to the basement columns.

The reinforcement of the floor slab consists of a layer of rectangular rods at the top and a layer of diagonal rods at the bottom, the latter having a slight lap. The rectangular rods lap over column heads with laps ranging from 5 ft. 4 in. to 8 ft. 4 in. This method is slightly different from the ordinary, and avoids four layers, common in most flat slab systems. The floor load was calculated at 125 lb. per square ft., and the mixture of concrete used at the column heads was quite rich, being 1 : $1\frac{1}{2}$: 2. It is stated that a saving in steel was made by this method.

The entire building was erected by the Concrete Engineering Company, of Boston, in accordance with plans and specifications of Monks and Johnson, the architects and engineers. The concrete was mixed on the ground in a 1-yd. Ransome mixer that discharged directly into Ransome dump buckets, which were hoisted to the top of the wooden tower, where the concrete was chuted directly to the forms.

A FIVE-MILLION-GALLON CONCRETE RESERVOIR.

The illustration forming the frontispiece to this issue shows the construction of a remarkable reinforced concrete reservoir built to ensure an auxiliary or emergency supply for the water system at Ashville, N.C., U.S.A. It has a capacity for 5,000,000 gallons of water, and measures 150 ft. in diameter at the bottom and is 40 ft. deep. The wall is $3\frac{1}{2}$ ft. thick at the bottom and tapers to a thickness of 8 in. at the top. The reservoir holds a quantity of water sufficient to last the city three or four days.

As first constructed the reservoir was not satisfactory, but was reconstructed and given a thorough test, the new work being done by Geo. H. Davidson, of Ashville, who laid an entire new floor 8 in. thick, using 900 barrels of Portland cement, twenty tons of $\frac{1}{2}$-in. reinforcing steel, 500 yds. of 1-in. broken stone and three tons of Trus. Con. waterproofing composition. The total cost of the reservoir was about £9,400.

REINFORCED CONCRETE WATER TOWER AND CHIMNEY IN SHANGHAI.

The water tower and chimney stack shown in the accompanying illustration were erected some time ago at Shanghai by the firm of Jardine, Matheson and Co., Ltd., for the Kiangsu Acid, Chemical and Soap Works.

The water reservoir has a capacity of 340 cubic metres (445 cu. yds.). Although the smoke in the chimney would not reach a very high temperature, in order to avoid any possible cracks in the bottom of the reservoir it was resolved to carry off the smoke-gases by means of a pipe which is absolutely separated from the reservoir. Also, to avoid cracks resulting from the difference of temperature between the chimney and the four columns supporting the tank, they were built independent of each other up to the top of the tank.

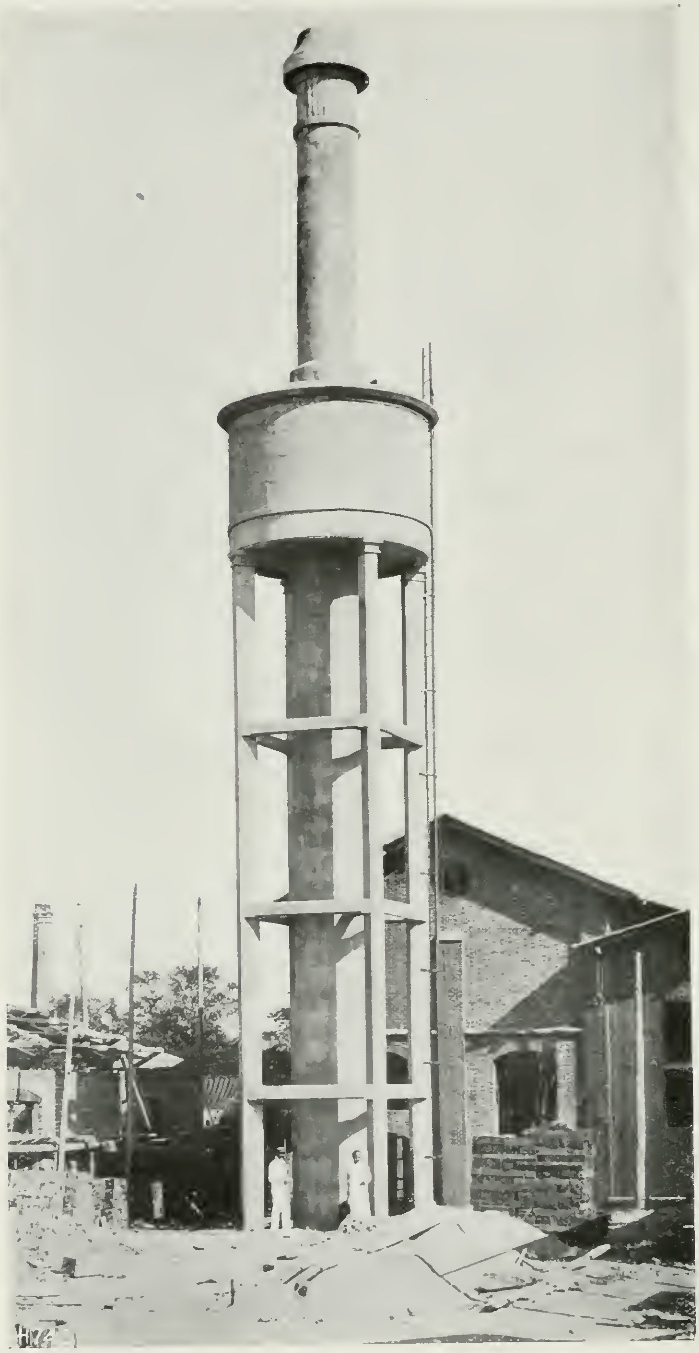
The chimney consists of two tubes leaving an air space between them of 3 in., which is connected at the lower part of the chimney with the outside air by means of four 6-in. inlets, so producing a draught of fresh air to reduce the temperature of the two tubes as far as possible.

The inner tube was built with wooden moulds, whereas the exterior centering of the outer tube consisted of riveted sheet iron, and the inner centering of the outer tube consisted of 1-in. concrete slabs, moulded three weeks before they were used.

The reinforcement consisted of Indented bars and round wrought-iron.

The cost of the combination of the chimney and water tank proved to be lower than two separate structures, besides reducing the building space to a minimum.

Since this structure was erected most of the buildings surrounding Shanghai were destroyed entirely by a large fire. One of them containing a large stock of sulphur prevented the firemen from taking prompt action. After the fire the water tank was tested and was found absolutely intact. We are indebted to Mr. C. Luthy, engineer, of Shanghai, for our particulars and photograph.

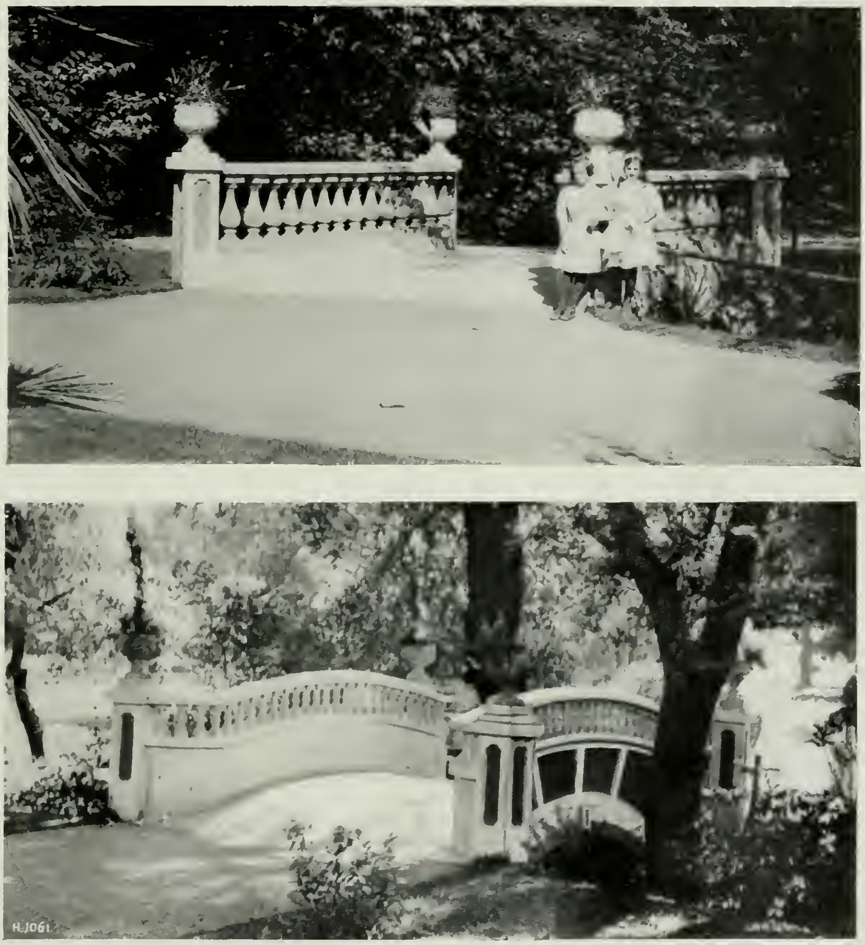


Reinforced Concrete Water Tower and Chimney Stack in Shanghai.

TWO REINFORCED CONCRETE BRIDGES IN THE PUBLIC GARDENS, HALIFAX, NOVA SCOTIA.

THE reinforced concrete bridges shown in the accompanying illustrations—from which their artistic appearance is plainly seen—were erected for ornamental purposes in the public gardens of Halifax, Nova Scotia.

That shown in the top illustration is a semi-circular arch having a span of 6 ft. The width of the bridge is 10 ft. 6 in. and the length 14 ft. The reinforcement consists of $\frac{1}{2}$-in. round bars placed at 6-in. centres. All balustrades were cast in cement moulds. The foundations are carried down 5 ft. 6 in. owing to the severe frost in the winter.



Views of the Two Bridges.

Two Reinforced Concrete Bridges in the Public Gardens, Halifax, Nova Scotia.

The lower bridge is an elliptical arch, with a span of 13 ft. The width is 10 ft. 6 in. and the length 15 ft. The arch is $7\frac{1}{2}$ in. thick at the crown. The reinforcement consists of $\frac{3}{4}$-in. round bars at 6-in. centres. The foundations are 2 ft. deep on solid rock. Both bridges are finished with plain white cement, rubbed smooth.

These bridges were designed by Mr. William E. Chesney, of Halifax, for the Parks Commission.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"Reinforced Concrete Design." By Oscar Faber, *B.Sc., A.M.I.C.E., A.M.I.E.E., A.C.G.I.*, and P. G. Bowie, *A.C.G.I.*

London: Edwin Arnold, 41-43 Maddox Street, W. Price 12/6 net. 8vo. Pp. 332+xx.

Contents.—Preface—Numbers of Principal Figures—List of Symbols—General Principles—Materials. Part I.: Calculation of Stresses under Known Forces and Moments- Simple Bending and Simple Compression—Bending Combined with Direct Forces—Adhesion and Shear. Part II.: The Design of Columns—Strength of Columns—The Determination of the Direct Loads on Columns—The Determination of the Eccentric Stresses in Columns. Part III.: The Design of Beams and Slabs—Beams—Slabs. Part IV.: Applications and General Notes—Reservoirs—Retaining Walls — Specifications — Quantities and Notes on Practical Applications—The Specialist Engineer and the Contractor. Appendix I.: Being Mathematical Analyses of Beams under Various Conditions of Loading and Fixing. Appendix II.: Second Report of the R.I.B.A. on Reinforced Concrete (1911).

There are none too many treatises on the subject of reinforced concrete by English authors, though we must consider at the same time that American authors can and do to a great extent help to supply the British market. Naturally, however, we welcome books that can compete with American works and with Continental books translated into English. We are pleased to record that the work here under review is a worthy rival. Its primary appeal is to the designer, and it discusses many problems that exercise the structural engineer who undertakes the design of reinforced concrete, which is so very different from the comparatively simple design of ordinary steelwork. The book will naturally be strong meat for elementary students.

The features of the book which deserve most praise are the original ones, and these are especially the simple and fairly accurate, though approximate, means of calculating the stresses in members under combined bending and direct stress, the detailed discussion of continuous beams, and the effect of unequal depression of supports, and also eccentric loading and flexure of columns due to beams built monolithic with them. The latter is the subject of a long appendix, and much space is given to it in the body of the book. It is certainly important, and as it is usually ignored or treated as indeterminate the authors have done well to deal with it thus fully. This and other elaborations in the treatment of reinforced concrete design are instructive as a general guide, but at the same time it would take far too much time, as well as being, after all, only an approximation, to warrant such detailed investigation in practice as a general rule—it is commonly met by what is called "judgment" and "standard practice" on the part of the designer.

Space precludes us from dealing with all the many interesting features of the book. Generally we think the authors have displayed excellent judgment and good commonsense, which have doubtless been assisted by their practical experience in design. In the present state of experience and experiment it is only to be expected that there should be much to be argued about and to dispute, and this book presents plenty of opportunity. Our only serious adverse criticisms are, however, that the treatment of shear is not at all adequate, and, indeed, is much in error; that often positive opinions are expressed without due explanation of reasons therefor; and that too strong a bias for or against particular methods of doing business in the reinforced concrete industry is suggested.

The book is exceedingly well produced, with many carefully drawn illustrations, and should be in the hands of all advanced students of the subject.

"Elementary Principles of Reinforced Concrete Construction." By Ewart S. Andrews, *B.Sc.*

London: Scott, Greenwood & Son. Price 3/- net.

Contents.—Introductory and Historical Note—Notation—Properties of Concrete and Steel—Loads, Working Stresses, Bending Moments, and Other Useful Data—Reinforced Concrete in Direct Compression—Reinforced Concrete Beams—Reinforced Concrete Beams Subjected to Bending Combined with Axial Thrust—Graphical Treatment of Reinforced Concrete Beams—Shear Stresses and Deflections for Reinforced Concrete Beams—Reinforced Concrete Columns—Some Leading Systems of Reinforcement.

Mr. Andrews is lecturer in theory and design of structures at Goldsmith's College, and in the present book has adopted an elementary standpoint and a manner of explanation specially calculated to appeal to the student, and to those who wish for some knowledge of this increasingly important subject without the necessity of an advanced mathematical knowledge. The author frankly explains that the method he has adopted consists in the explanation of the theoretical treatment of a problem according to certain assumptions, and in illustrating this by numerical examples, the same example being sometimes worked out by several methods. For a lecture text-book this method is admirable; for the more difficult work of the private student it would have been better to include more examples and to work them out somewhat more fully. Considering its small size, however, the book is well written, and should be easy to follow, especially as the author has marked some of the more difficult portions so that they may be omitted in a first reading.

The author rightly insists that before we can study the method of construction known as reinforced concrete we must be very clear as to what is meant by this term. Yet he defines it in the following somewhat indefinite terms : " A reinforced concrete structure is one in which concrete and a reinforcing material (usually steel) are so combined as to act together as one material, the two materials being so placed relatively to each other as to produce a more economical material than either acting alone."

The reader used to the term will at once picture to himself an idea which will coincide exactly with this definition, but the one who comes to the book with no knowledge of the subject will, in most cases, fail to form any clear conception of the meaning of the term. He may, of course, glance through the illustrations without reading any further, and so may gain the knowledge he desires ; but it would surely have been infinitely better to have inserted a diagram and to have repeated the definition in terms of the illustration. In the present case this is all the more desirable, because in the sentence immediately following his definition the author describes a form of construction which he wishes to treat as an exception, thereby adding more confusion to the mind of the unsophisticated student.

The " history " of reinforced concrete is confined to three small pages, of which nearly a page is devoted to the " future " ; there is a list of the notation used occupying five pages, and we are pleased to see that this notation is that recommended by the Concrete Institute. After this the author gets to work in earnest, and the rest of the book is straightforward " business," the information and explanations being given in clear and comparatively simple language, though the author seldom, if ever, descends from his academic rostrum to " sit by the side " of his pupil and to explain to him in the manner adopted by all the greatest teachers with those students who are really keen to learn. The result is a compression of style which is disconcerting to the beginner who is unable to attend lectures, or who has no one to whom he can go for further explanations or to clear up some trivial yet for the moment difficult point.

In spite of these minor drawbacks—which may be remedied in a future edition—this little book is one which should be studied carefully by those who want a good grounding in the subject. To such we can recommend it heartily.

CORRESPONDENCE.

Under this heading we invite correspondence.

Tenders for Reinforced Concrete Work.

Dear Sir,—We have frequently noticed that in competitive schemes and tenders for works in reinforced concrete there is an enormous difference between the amounts of the various tendering contractors, and generally the work is given to the lowest tender. As it appears to us that it is impossible that these considerable differences between the various tenders should be due to the more or less economy of the competing systems, we have made inquiries to ascertain the reason of these discrepancies, and we have found that practically in all cases these were due to the fact that the quantities of steel, concrete, and centering given by the various firms of designers showed considerable differences which could only be explained by the fact that either different methods of calculation had been used, or a different factor of safety, or finally different conditions of loading, in spite of a clear specification having been issued by the architect or engineer requiring tenders.

We are strongly of opinion that, in all fairness to the various competitors, the architect or engineer who is to choose between the competitors should, before finally accepting the lowest or any of the contractors, investigate his calculations and his design, and in particular the bill of quantities. In this manner the work would be given to those firms who have made a proper study of the scheme, and all discussions between the architect or engineer and the contractors concerning insufficient quantities will be settled before the work is actually given to any particular firm, and this will save any discussion which may arise later on during the execution of the work concerning the insufficiency of the quantities, and incidentally the work will have been placed in proper hands.—Yours faithfully,

The Editor,
Concrete and Constructional Engineering.

Edmond Coignet, Limited,
G. C. Workman,
Managing Director.

The Design of Reinforced Concrete Columns.

39, Newhall Street, Birmingham, April 10th, 1912.

Dear Sir,—Permit me to welcome Mr. Marsh's letter of March 24th, coming as it does from one so well able to speak on the subject.

The whole paragraph on page 17 of the Committee's second report reads: "The spacing factor should not be taken at more than 32, even if p is less than $\cdot 2d$, but *intermediate* values of the spacing factor may be obtained from the equation:

$$s = 48-80\,\frac{p}{d}$$

Clearly, the increase in permissible stress in the concrete is proportional to s. If, however, the maximum value of s is fixed, what is the good of taking p less than $\cdot 2d$, for it will only have the effect of making s greater than 32? For when $p = \cdot 2d$

$$s = 48-80x : \frac{2d}{d}$$

$$s = 48-16$$
$$= 32$$

With regard to Mr. Marsh's second point. It is quite true that the Committee recommends that—

"The stress on the metal reinforcement (*i.e.*, the value of *m.c.*) should not exceed 0·5 of the yield point of the metal."

It is also quite true that "for a mild steel with a resistance at yield point of 32,000 lb. per sq. in., the allowable stress would be 16,000 lb. per sq. in. . . ."

Mr. Marsh will be quite aware that it is possible to get, with special steels, a much higher resistance at yield point than 32,000 lb.; in fact, twice that amount is readily obtainable. The whole point, I suggest, is: What is a good, general, and safe figure to be used with the ordinary steel to be obtained on the market?

The working stress used in constructional steelwork for steel with an ultimate strength of 28 to 32 tons per sq. in. is $7\frac{1}{2}$ tons, or 16,800 lb. per sq. in. It seems to me, therefore, that Mr. Marsh might have taken a higher figure in his illustration, as might also the Committee in their recommendations.

Those of us who have to design structures to pass the requirements of local authorities know that they are willing to accept designs based on the R.I.B.A. Committee's Report. If the recommendations had not been quite so conservative, we should have been able to save our clients some expense without running any risk.

Again expressing my pleasure that Mr. Marsh has used my article to ventilate the views of the Committee, I am, dear Sir, yours faithfully,

(Signed) HARRY JACKSON.

P.S.—It would be very instructive if Mr. Marsh would indicate how the formula $s = 48-80\frac{p}{d}$ was arrived at.

Mr. Marsh's Reply to Above.

April 18th, 1912.

DEAR SIR,—I beg to thank you for sending me a copy of Mr. Jackson's letter of April 10th.

The advantage in using a closer spacing than $0{\cdot}2d$ for the lateral bindings accrues from the increase in the value of r in the equation $c = cp\ (1 + fsr)$. After fs reaches a value of 32 no increase may be made to this factor, but the value of r may be increased to as great an extent as may be deemed advisable. The equation $s = 48 - 80\frac{p}{d}$ is a straight line equation obtained from the values of s given in the report. It might have been equally well stated that intermediate values could be obtained by interpolation of arithmetical means.

With respect to the stress on the metal reinforcement, the Committee recommended that, for beams, the maximum allowable stress should be 16,000 lb. per sq. in. for mild steel, and that if stronger steel were used, the stress should be $0{\cdot}5$ of the stress at yield point, but such stress should not exceed 20,000 lb. per sq. in. The reason for this recommendation was that a limit must be placed on the allowable tensile stress on the reinforcements to prevent any dangerous cracking of the concrete on the tensile surface.

In the case of compression members no dangerous cracking was to be feared, and the Committee therefore recommended that in the case of all qualities of steel used for the longitudinal reinforcements the allowable stress should be taken as $0{\cdot}5$, the resistance at the yield point.

In my diagram for the calculation of compression members this is clearly shown by the introduction of horizontal lines as the limits for steels having various resistances at the yield point. Yours faithfully,

The Editor, Concrete and Constructional Engineering. CHARLES F. MARSH.

MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Sixty-ton Test on Concrete Pile.—A remarkable test on a concrete pile was made during the course of last year by Lockwood, Greene & Co., of Boston, on a "Simplex" concrete pile driven at the New Print Works of the Pacific Mills by the New England Foundation Co., the contractors. Although the duration of the test was two months, and the load 60 tons for more than one month, the settlement recorded was practically negligible. It is undoubtedly one of the most exhaustive tests ever made on a concrete pile; and it should be noted that the pile was driven in the spring of the year, when the ground was moist and cold, causing the concrete to set comparatively slowly. The figures have added value, as data on this subject are extremely scarce, principally due to the fact that there is not usually an opportunity for testing a pile which will see actual service. However, conditions at the Pacific Mills are such that a complete test was possible.

The general method used in the "Simplex" system consists in driving to a suitable bearing, under a drop hammer operated in a substantially rigged pile-driver frame, a 16-in. diameter, $\frac{3}{4}$-in. thick hollow cylindrical steel tube, or form, to the lower end of which is fitted loosely a conical cast-iron point. As practically the entire energy of the hammer is delivered to the point, full value is given to the penetration, equal compression to the soil all the way down, and developing the full bearing value of the soil from top to point. Upon reaching the required depth the form is filled with soft concrete, by means of drop bottom buckets, to a sufficient height and quantity to fill the pile hole to proper level. The form is then pulled out, leaving the point in the ground, the concrete being forced down and out by its own weight, thereby completely filling the hole to its compacted wall. The concrete is always placed before the removal of the form, no shell being left in the ground.

The pile under test was driven on the banks of the Merrimac River, through a stratum of gravel which is filled with water. In order to keep the water out of the tube while it was being filled with concrete a coil of oiled jute was placed between the cast-iron point of the pile and the steel tube. To obtain an even bearing area for the load the pile was cut off, capped, and then surmounted by an iron plate, upon which were balanced 8-in. × 14-in. timbers, over which were placed similar timbers at right angles. The platform was then erected to support the brick wall.

By counting three different piles of 100 bricks each the average weight per brick was found to be 5·18 lb. Every part of the platform and all materials supporting the brick load were carefully weighed previously and found to weigh 3,456 lb.

Readings were obtained by means of a piece of $1\frac{1}{4}$-in. gas pipe, 17 ft. long, placed upon the iron plate between the timbers, and by referring to a nailhead placed in the river wall near the pile and assumed as zero. Readings were taken daily at nearly the same hour, corrections of temperature for above gas pipe being made after June 2nd.

The test began on May 13th, which was ten days after the pile was driven. A load of 30 tons was applied on the 14th, and this was not changed until May 19th, the difference of readings being ·006. On the latter date 15 tons more were applied, making the load 45 tons. On June 8th the load was finally increased to 60 tons, and this was left until July 14th. The total variation from May 14th to July 14th, when the test was closed, was ·012, which was the apparent settlement.

The diameter of the pile was 16 in., and its length 19 ft. 7 in. The concrete was mixed in the proportion of 1 : $2\frac{1}{2}$: 5.

Le Chatelier's Test for Constant Volume.—At the recent annual meeting of the German Association for Testing Materials, Dr. O. Strebel reported the results of an extensive investigation on the Le Chatelier test. This well-known test consists in measuring the expansion of a cylinder of neat cement by means of a brass cylinder 30 mm. diameter and 30 mm. high, provided with two long indicating limbs. These indicating limbs are 165 mm. in length from their pointed tips to the centre of the cylinder.

The test is made by laying the appliance on a glass plate and filling it with cement mortar, the mould being kept closed by means of a clamp. A second glass plate is then laid on top of it and is loaded with a convenient weight. The whole apparatus is next immersed in cold water at room-temperature (56° F.), and is allowed to remain for twenty-four hours. On removal, the distance between the indicating limbs is measured, the apparatus again immersed in water, and the temperature of the latter is then raised to boiling point in twenty-five to thirty minutes and is kept boiling for six hours. At the end of this time, the apparatus is removed from the water, allowed to cool, and the distance between the tips is again measured. The increased divergence should not exceed 10 mm.

The Committee on Accelerated Tests of the International Association for Testing Materials presented to the Copenhagen Congress in 1909 (see *Concrete and Constructional Engineering,* 1909, IV., 349), recommended that the expansion in cold water should also be measured, but no figures for this were reported by Dr. Strebel.

Dr. Strebel began by pointing out the irregular results obtained by various investigators and the general unwillingness to accept the indications of this test, these culminating in the Stuttgart meeting of the association at which a new series of tests was ordered. He then dealt with a recommendation in which it was suggested that all cements should be ærated twenty-four hours and seven days, respectively, before being tested. Six cements were tested in ten different laboratories—(*a*) as received, (*b*) after twenty-four hours' æration, and (*c*) after seven days' further æration. The cements were purchased in the ordinary manner, those which were selected being suspected of a tendency to expand. They were analysed and tested as shown in the accompanying table :—

Analysis and Tests.

	Cement 1.	Cement 2.	Cement 3.	Cement 4.	Cement 5.	Cement 6.
Chemical Analysis :						
Insoluble matter	2.48	0.83	0.41	0.24	0.64	1.66
SiO_2	20.39	19.94	18.24	20.37	21.13	19.46
Al_2O_3	10.41	8.46	7.31	7.32	6.37	7.94
Fe_2O_3	2.53	2.78	2.74	2.63	3.61	3.45
CaO	52.48	61.71	62.07	61.82	64.13	58.91
MgO	2.59	1.65	1.87	1.85	1.52	3.43
SO_3	3.36	2.18	1.67	1.69	1.06	2.01
Sulphides as S.	0.29	0.01	0.02	0.06	—	0.09
Loss on Ignition	4.12	2.58	4.48	3.34	2.54	2.54
Specific Gravity :						
Raw	2.941	3.027	2.948	3.014	3.133	3.045
Calcined	3.125	3.157	3.193	3.189	3.205	3.205
Litre weight in kg., loose	1.008	1.081	1.056	1.149	1.211	1.039
After shaking	1.599	1.726	1.636	1.816	1.920	1.746
Per cent. residue on sieve :						
No. 75	1.2	1.0	0.5	1.1	0.6	1.8
No. 175	16.2	18.0	11.5	17.6	19.4	21.4
Setting Time :						
Beginning	4 Hrs. 25	4 Hrs. 35	3 Hrs. 45	3 Hrs. 45	3 Hrs. 55	2 Hrs. 20
End	8 Hrs. 25	7 Hrs. 30	6 Hrs. 45	7 Hrs. 35	6 Hrs. 25	6 Hrs.
Strength 1 : 3 normal sand (after immersion) in kg/qcm :						
Tensile, 28 days	21.4	27.5	25.7	27.3	22.4	23.3
Crushing 7 days	113	288	253	181	212	166
28 days... ...	157	373	322	234	366	234
(After combined storage) in kg/qcm :						
Tensile, 28 days	36.9	47.3	42.2	40.2	38.5	38.5
Crushing, 28 days ...	214	443	375	294	407	281

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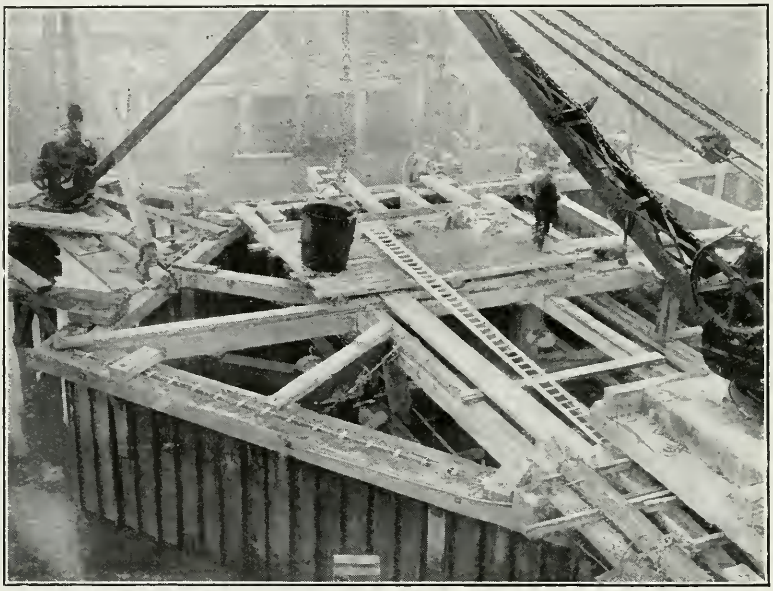

Illustration shows one of three cofferdams constructed with our steel piles at L.C.C. Reconstruction, Deptford Creek Bridge.

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Used by the BRITISH AND COLONIAL GOVERNMENTS, FRENCH GOVERNMENT, AND LEADING CONTRACTORS AT HOME AND ABROAD.

Latest Types of Pile-Driving Equipments

Cement 1 showed a relatively small expansion after boiling. The fresh cement and that after twenty-four hours' æration seldom expanded more than 5 mm. Curiously enough, at one laboratory the sample, after seven days' æration, exceeded the limit just mentioned, and in this one instance proved erratic. The same sample also behaved curiously whilst setting. The expansion in all three periods of æration showed very variable results in the hands of different chemists. In one laboratory the expansion was reported as less than 1 mm., but in three others as over 10 mm. In short this cement proved to be abnormal and irregular during the early stage of setting, but later proving quite constant in volume. This cement is one in which the expansion tests have ordinarily been made in cold water. The high percentage of sulphur trioxide shows that it contains gypsum—a well-known cause of expansion.

Cement 2 showed extraordinary variations. When fresh, the smallest expansion was 4 mm., the largest 37 mm. After twenty-four hours' æration the smallest expansion was 3½ mm., the largest 31½ mm. In one laboratory the conventional limit of 10 mm. was not exceeded, and the cement reported as constant in volume. After seven days the smallest expansion was 2 mm., the largest 20½ mm. In two cases the cement was reported as constant in volume after this period.

Cement 3 behaved similarly to cement 2, but the variations were somewhat larger on account of a more tardy setting. It gave the following results :—

	Smallest Expansion.	Greatest Expansion.
Fresh Cement	4mm.	25mm.
After 24 hours' æration	6mm.	23mm.
After 7 days' æration	5½mm.	21½mm.

Here again Laboratory I gave the lowest results—viz.,

Fresh cement 4 mm. expansion.
After twenty-four hours' æration 5½ mm. expansion.
After seven days' æration 6 mm. expansion.

These are the mean of duplicate tests.

Laboratory F also reported that the cement was within the limits of the Le Chatelier test.

Cement 4 also showed great variations. Laboratory I reported "constant volume," whilst E reported as high an expansion as 33 mm. The expansion in cold water showed a similar result. It is noteworthy that two chemists reported the highest expansion, whilst another reported that the cement had a constant volume.

Cement 5 was a rapid setter in hot water, and it exceeded the ordinary limits of the Le Chatelier test, the results also varying greatly among themselves.

The following results are interesting :—

	Smallest Expansion.	Greatest Expansion.
Fresh Cement	15½mm.	27mm.
After 24 hours' æration	25mm.	71½mm.
After 7 days' æration	14mm.	53mm.

This cement possesses the curious property of showing little or no expansion during the first twenty-four hours' immersion in cold water. The differences of the results obtained by different laboratories agreed better than those for some other cements, but here again E and H reported the highest and F and I the lowest expansions.

Cement 6 was reported by Laboratory I to be within the prescribed limit (10 mm.), and the others reported it as outside the limit. The following results were obtained :—

	Smallest Expansion.	Greatest Expansion.
Fresh Cement	5mm.	22½mm.
After 24 hours' æration	8mm.	22mm
After 7 days' æration	7mm.	20½mm.

The great variations in the results compel the conclusion that the Le Chatelier test is unreliable. This has recently been confirmed, said Dr. Strebel, by tests in English laboratories. Denmark in its newly-issued Standard Specification retains the cold water test and only admits the Le Chatelier test under limitation, in spite of the fact that it was Danish results which first caused this test to become popular.

Dr. Strebel concludes :—

(*a*) That the Le Chatelier test of a cement which shows a tendency to expand in cold water does not expand. If a cement has an abnormal composition it is soon taken off the market, and a person who relies on the Le Chatelier test will experience trouble.

(*b*) That the same cement gives widely differing results in different laboratories. The danger of using the test under these circumstances is obvious. Even with highly skilled and most painstaking chemists the results are discordant, and when the test is carried out in the ordinary course of works routine the results would be most misleading.

EDITORIAL MEMOS.

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DATES OF ISSUE.—This Journal is issued Monthly. For Advertisement Rates apply to Publisher. Matter for displayed advertisements required by 12th of month prior to subsequent issue, for Wants column by 18th.

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General Offices:
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Waterloo Place, London, S.W.

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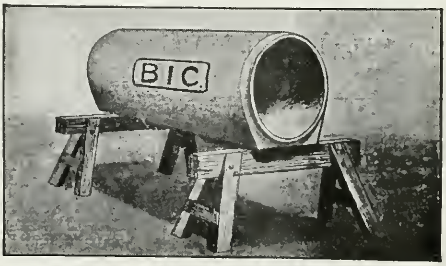

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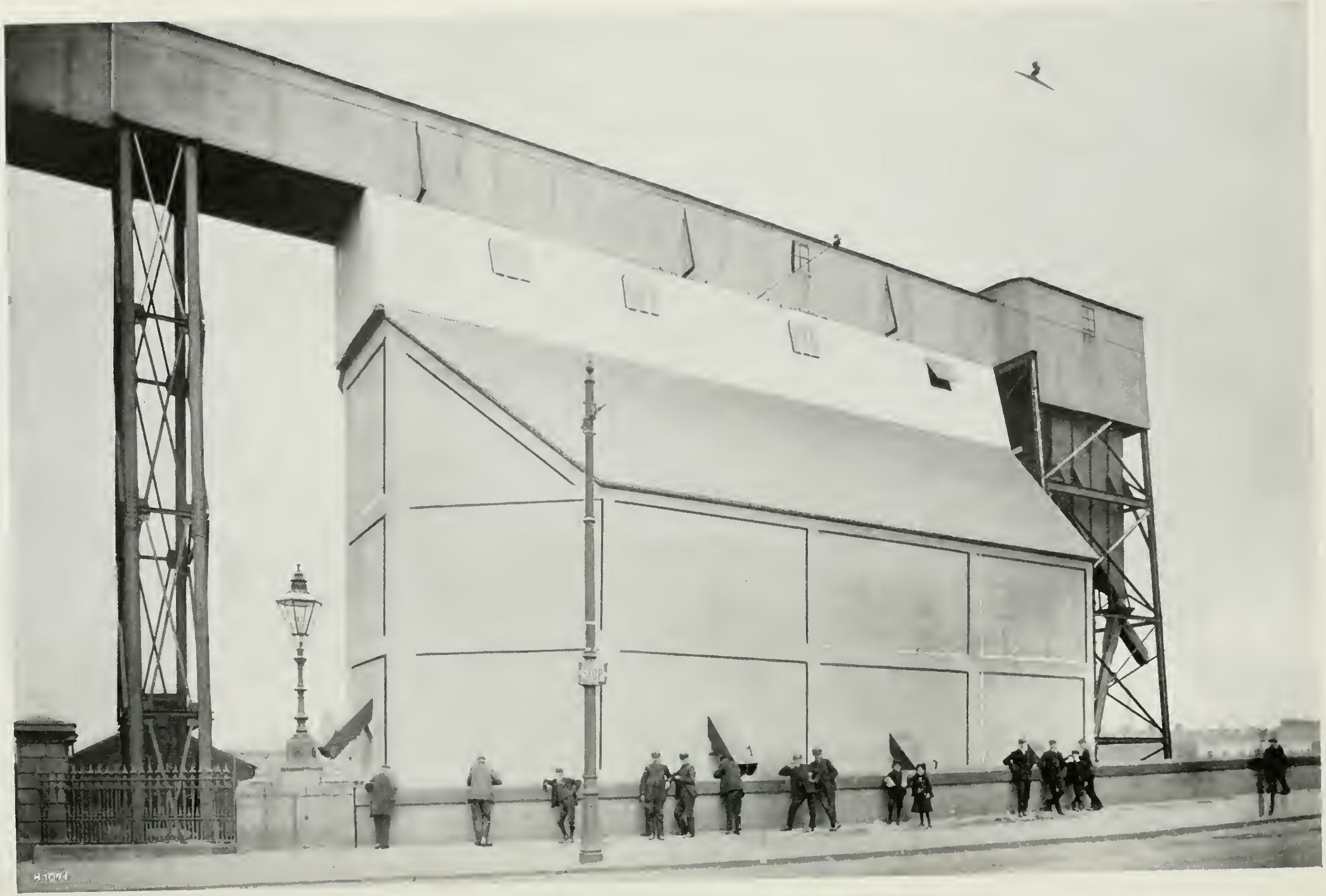


(*For description see page* 471.)
REINFORCED CONCRETE COAL BUNKERS FOR BELFAST CORPORATION.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 6. LONDON, JUNE, 1912.

EDITORIAL NOTES.

THE CONCRETE INSTITUTE.

THE Concrete Institute has again completed an official year of its existence, and has marked the event by holding an annual general meeting. There has also been the anniversary dinner, which has now become a function of the year, and, generally speaking, these two events have suitably marked the conclusion of its session.

This journal has done everything possible to assist the Institute during its inception and early days, and if it has occasionally had to publish strictures as to a certain absence of progress and development, and has criticised some of its work, it is, we believe, realised on all hands that this was not done in any carping spirit, but with a view to helping the Institute in its career, and at the same time assist in the general scientific, technical, and practical development of the uses of concrete.

With the Annual General Meeting referred to above there was presented the annual report of the Institute for the session 1911-12, and, whilst it contains the general features to be found in all societies' reports, there is certainly no doubt that the Institute during its last twelve months has done some very useful, tangible, and effective work. That it should have been able to do this during a period when much of its time was devoted to the discussion of its scope, its rules and future, is a matter for congratulation, for to the outside world and to the general membership it appeared as if these discussions on administrative questions were given precedence to the discussion of scientific and practical work.

REGULATIONS FOR REINFORCED CONCRETE.

Perhaps the most useful piece of work that has been done by the Institute in the 1911-12 session was in connection with the proposed regulations for reinforced concrete under the London County Council General Powers Act, 1909, without being coerced by the antagonistic policy towards these regulations advocated by certain other societies, and mainly by the Royal Institute of British Architects. The Concrete Institute has done good work in pointing out certain features that can be improved, and where suitable compromises might be arrived at between divergent views. We hope that in future, in matters relating to legislation, the Institute may rather work in the direction of standing on its own feet, and precisely putting forward its own views, without giving much time to the various discussions of other bodies, none of which can have such a comprehensive knowledge of the subject as the members of the Concrete Institute, and none of which can put forward its case against legislation without obvious bias. One of the objects of the Concrete Institute's existence is to obtain sound and fair building, and such sound and fair building cannot be obtained without regulations, as human nature is not so prone to safeguards as to apply them fully without compulsion.

With regard to the work of the committees, we would particularly refer our readers to the report, from which it will be seen that the scope of the subjects under review is vast, and very great credit will be due to the committees in question if they can bring feasible conclusions before the membership on some few of the many problems they have in hand; but we think that it would be as well for the Institute to devote its energies to two or three immediate problems rather than to fritter away its energy with two or three each dealt with in a leisurely fashion.

That the Institute is referred to in an Act of Parliament on matters relating to concrete, and that it should be invited to send representatives to joint commissions, is flattering to such a young institution. But we would suggest here that the work of representing the Institute on other bodies should be distributed, as far as possible, amongst the members of council or members of standing committees, so that not too great a load should fall on any one man.

MEETINGS.

Of administrative features, we welcome the more frequent issue of the Institute's "Transactions," and we are glad to observe that the library is making some progress. But what is wanted more than anything else on the administrative side are really lively meetings, with good debates, and these have been somewhat lacking during the past year. We are given to understand that the programme for the next year will show certain improvement in this direction. Some of the papers presented before were really excellent, but the discussion in most cases was poor indeed. We have frequently expressed our view as to the locale of the meetings, but, assuming that this cannot be changed, we think much could be done by having an active subcommittee to make the arrangements for the meetings, invite the speakers to the discussions, and the like.

Turning further to the report, we find a slight increase in the membership, and we think, with energetic management, that this should continue to increase, although we are not forgetting that this membership of some 900 is a very strong one for so young a society.

CHANGES ON THE COUNCIL.

Regarding the retirement of some prominent members of the council, which we also notice in the report, we naturally realise that some of the older and distinguished engineers, who really only joined to assist the venture, to add prestige to its infant years, and give advice when called upon, could not be well expected to attend meetings. Therefore, replacement by younger and more active men, now that the Institute is well on its way, is perhaps an advantage. Nevertheless, it should be remembered that the ladder which the Institute climbed was supported by such eminent men as Sir Douglas Fox, Sir William Preece, Sir William Mather, and Mr. Alexander Ross, and that very great thanks are due to these for the early assistance rendered to the Institute. That Mr. William Dunn, whose name has been very prominent in matters relating to concrete, has also retired is certainly to be regretted in one sense. On the other hand, his well-known antagonism to reinforced concrete regulations, in which he figured so largely in the Royal Institute of British Architects, put him very much out of sympathy with the general interest of the Institute, and his replacement by so eminent an authority as Mr. Boulnois—who was till lately connected with the Local Government Board—is again an advantage to the Institute. Nevertheless, it should here be remembered that Mr. William Dunn rendered eminent service to the Institute's Science Committee, and that the thanks of the Institute are due to him for much useful work in that direction.

RETIREMENT OF SIR HENRY TANNER.

We now turn to another administrative change. Sir Henry Tanner has concluded his two years' office as President, but we are pleased to observe he retains his seat on the council and an active interest as Past-President. We think this is a suitable opportunity to point out to our many readers the very many services rendered by Sir Henry Tanner to the Institute during his two years of office. The Institute has not grown in numbers and funds during that period, but it has gained immeasurably in prestige, which was in some measure, if not entirely, due to Sir Henry Tanner. This particular feature of the Institute's existence under Sir Henry Tanner's reign cannot be underestimated, as a certain amount of internal dissension seemed likely to weaken the Institute at a critical period. Had it not been for Sir Henry's presidency during that critical period, there is, to our mind, not the slightest doubt but that the Institute would have materially decreased in prestige, and it might have eventually led to its being split up into two minor societies, representing different interests. The great courtesy and justice of Sir Henry Tanner's decisions in the Institute's career practically minimised internal trouble, smoothed over the difficulties, and made it possible for him to leave the chair at the end of his term with an Institute higher in prestige than the one in which he had taken office, and an Institute which seems to have a bright future before it.

We feel sure that the primary feature of the Institute's work during the next year should be activity in practical and instructive matters, and that it should be oblivious to internal administrative matters. Whatever the Institute's report on scope may be, or, rather, what the findings of its committee may be, as to titles or sub-titles, these are entirely matters of minor importance compared with successful records of work done and accomplished. If under the new President's rule a good record can be obtained of work achieved, it will be a great feather in his cap; for, whilst there is much to do, there is no doubt the whole subject of concrete is such a complex one, so subject to difference of opinion, that tangible results are difficult to obtain.

THE NEW PRESIDENT.

The new President, it should be mentioned here, is Mr. E. P. Wells, J.P., who took a most active part in the formation of the Institute, and who has acted as honorary treasurer in a most judicious manner. We wish Mr. Wells every success on the new appointment, and we congratulate him on holding the highest office in the profession of concrete designing, which he adorns. The position which he will now occupy for two years is no easy one, but it is an eminently distinguished one, and if, as we trust, the Institute again progresses—be it in prestige, in numbers, or in funds—there is no doubt a large amount of credit for such achievements will be due to him who occupies the chair.

Sir Henry Tanner, C.B., I.S.O., F.R.I.B.A.

THE CONCRETE INSTITUTE.

ANNUAL GENERAL MEETING.

The Third Annual General Meeting of the Concrete Institute was held on Thursday, May 9th, 1912, at 4.30 p.m., at Denison House, Vauxhall Bridge Road, Westminster, S.W., Sir Henry Tanner, C.B., I.S.O., F.R.I.B.A., presiding, when the Annual Report was put forward. Below we give a summary of the report and of the proceedings.—ED.

REPORT OF COUNCIL FOR 1911-12 SESSION.

THE Concrete Institute had on April 30th, 1912, 881 Members, 24 Students, and 11 Special Subscribers.

The increase in membership from the foundation of the Institute is recorded in six-monthly periods in a chart accompanying the report.

The finances of the Institute are, as shown by the balance-sheet, in a satisfactory condition.

The general meetings and visits are duly recorded from time to time in the "Transactions," and therefore are not included here. The thanks of the Institute are due to the authors of the papers and to those who acted as hosts on the occasions of the visits referred to.

The Council has in the past year been concerned with numerous technical and administrative matters.

One of the principal items occupying the attention of the Council and a Joint Committee of the Science and Reinforced Practice Standing Committees has been the consideration of the proposed Regulations made under the provisions of Section 23 of the London County Council (General Powers) Act, 1909, with respect to the construction of buildings wholly or partly of reinforced concrete.

A draft of suggested Regulations was submitted by the Superintending Architect of the London County Council to the Institute for its consideration, and the action taken was recorded in the annual report for the 1910-11 session. The 1909 Act referred to provides that the Concrete Institute, together with the Institution of Civil Engineers, the Royal Institute of British Architects, and the Surveyors' Institution, shall have notice of the Council's intention to apply to the Local Government Board for the allowance of any regulations as to the use of reinforced concrete in the county of London, and such notice was duly given at the beginning of December, 1911. The Institute's suggestions as to the amendment of these regulations have been sent to the Secretary of the Local Government Board and to the Superintending Architect of the London County Council, and in due course the Council of the Concrete Institute will be able to inform the members as to the outcome of such action.

In October, 1911, a proposal for widening the scope of the Institute was considered by the Council and referred to a Committee, whose report was approved on March 14th, 1912; an abstract of the report is appended. The members of the Committee are: Mr. E. F. Etchells (Chairman), Professor Henry Adams, Mr. Alexander Drew, Mr. Charles F. Marsh, Mr. W. G. Perkins, Mr. Edwin O. Sachs, Mr. H. D. Searles-Wood, Mr. L. Serraillier, Sir Henry Tanner, and Mr. E. P. Wells.

A seal, medal, die, and certificate of membership have been designed and engraved for the Institute by Mr. Cecil Thomas, and the certificates of membership have been issued to all the members who have paid their subscriptions for the current year.

The question of awarding a bronze medal for the best paper read in the previous session was decided by ballot among the members of Council, and as a result the medal has been awarded to Professor Beresford Pite, F.R.I.B.A., for his paper, entitled "The Æsthetic Treatment of Concrete."

Sir Henry Tanner has been appointed delegate of the Concrete Institute at the Congress of the Royal Sanitary Institute to be held in June next.

The International Association of Road Congresses have asked the Institute to appoint delegates to the Third International Road Congress to be held in London in June, 1913, and delegates will be appointed after the annual general meeting has taken place.

The following members of the Council resigned in the past year: Mr. William Dunn, Captain J. Gibson Fleming, Sir Douglas Fox, Mr. W. T. Hatch, Mr. W. H. Johnson, Mr. F. May, Sir William Preece, and Mr. Alexander Ross. The Council co-opted the following to fill some of the foregoing vacancies: Mr. H. Percy Boulnois, Mr. Alexander Drew, Mr. A. Alban H. Scott, and Mr. L. Serraillier.

In consequence of the resignation of Sir Douglas Fox and Sir William Preece as Vice-Presidents of the Institute and of the retirement of Sir William Mather, Mr. William Dunn, Mr. C. S. Meik, and Mr. F. E. Wentworth-Sheilds were appointed to the vacancies in November. Mr. William Dunn and Mr. Alexander Ross resigned as Vice-Presidents in March, and Mr. H. Percy Boulnois and Mr. E. P. Wells were elected in their stead.

Sir Henry Tanner's term of office as President expiring in May, Mr. E. P. Wells was appointed President for the ensuing two years. The Council is pleased to record that Sir Henry Tanner will continue as a member of Council in the capacity of a Past-President and continue to give the Institute that help and guidance which have been so valuable in the past.

It has been decided that Chairmen of the various Committees shall be subject to annual election or re-election, and that the lists of members of the Standing Committees shall be revised after each annual election of members of Council. It has been decided also that Chairmen of the Committees shall be *ex-officio* members of the Finance and General Purposes Committee.

The First Annual Dinner of the Institute was well attended, and passed off successfully. The attendances at the two days' Summer Meeting were not satisfactory, and it has not been thought advisable to hold similar meetings this year.

During the present session a larger number of meetings will have been held than in any previous session, and next session it is intended to resume the course of Educational Lectures to students which were begun last session. The work of the Institute is increasing, and it is hoped to extend the number of meetings next year, and that the subjects to be dealt with will be of more general interest by reason of the extension of the scope to cover structural engineering generally.

A library is in process of formation at the Institute, and a number of donations have been received; but the Council would welcome the presentation of books, papers, and other publications on the subject of structural engineering generally, the funds at present precluding any large purchase of such works. A catalogue of works in the library will eventually be published, and members will be enabled to borrow books.

The "Transactions" will in future be issued quarterly, and articles or notes from members will be welcomed for insertion therein. In order to meet the extra cost it is proposed to invite advertisements, and the assistance of members in this direction will be valuable.

The Finance and General Purposes Committee have held regular meetings preliminary to each Council meeting, and the general results of their deliberations are recorded in the foregoing particulars of the work of the Council during the year. Mr. H. D. Searles-Wood and Mr. G. C. Workman were appointed members of the Finance and General Purposes Committee in June, 1911, in lieu of Mr. W. T. Hatch and Mr. L. Serraillier.

As regards the work of the Science Standing Committee, the Institution of Municipal and County Engineers requested the opinion of the Concrete Institute upon a Standard Specification for Concrete Flags which they proposed issuing, and the Committee made a number of suggestions which the Institution has intimated will be considered in any revision they may make in the future of their Specification. The Standard Notation for calculations for reinforced concrete which was drafted by the Science Committee and approved by the Council continues to meet with favour, and has been adopted by the Joint Committee on Reinforced Concrete appointed by the Royal Institute of British Architects in their Second Report, and by the London County Council in their proposed regulations covering the erection of buildings of reinforced concrete in London. Authors of textbooks have also employed it, and the American Joint Committee on Concrete and Reinforced Concrete have informed the Science Committee that they have the Notation under consideration with a view to its adoption in any further report they may make. Professor Henry Adams and Mr. E. F. Etchells were appointed Chairman and Vice-Chairman respectively of the Science Standing Committee in June. Mr. F. E. Wentworth-Sheilds having resigned as Hon. Secretary of the Committee, Mr. H. K. G. Bamber was appointed in his stead. The Committee have the following matters under consideration :—

(1) Reinforced concrete piles.
(2) A chemical test for steel used in reinforced concrete.
(3) The standardisation of attachments or joints in reinforced concrete.
(4) The adhesion of or friction between concrete and steel.

(5) A standard notation for calculations in structural engineering generally.
(6) A standard specification for reinforced concrete work.
(7) The effect of sewage on concrete.
(8) The effect of oils and fats on concrete.

The Science Committee appointed the following as representatives of the Institute upon the Joint Committee on Loads on Highway Bridges which is conducted by the Concrete Institute: Professor Henry Adams, Mr. William Dunn, Mr. Charles F. Marsh, and Mr. C. S. Meik. The other members are as follows:—

John A. Brodie, M.Eng., Wh.Sc., M.Inst.C.E.	Representing the Institution of Municipal and County Engineers.
A. E. Collins, M.Inst.C.E.	
J. W. Cockrill, M.Inst.C.E.	
A. E. Prescott, M.R.San.I.	Representing the Institution of Municipal Engineers.
Henry C. Adams, Assoc.M.Inst.C.E., A.M.I.Mech.E., A.M.I.E.E.	
H. C. H. Shenton, M.R.San.I.	

The Joint Committee have had one meeting and have issued inquiry forms to municipal authorities and others soliciting information respecting the vehicles that have to be sustained in various parts of the country as a preliminary to drafting recommendations.

The Tests Standing Committee have issued reports on "The Testing of Concrete, Reinforced Concrete, and Materials Employed Therein" and "The Testing of Reinforced Concrete Structures on Completion." These have been discussed at ordinary general meetings of the Institute. The Committee have the following matters under consideration:—

(1) The effect upon steel of the presence of sulphur in aggregates.
(2) The grading of aggregates.
(3) The expansion and deterioration of concrete due to changes of atmospheric temperature.
(4) The effect of the use of sodium silicate on the surface of concrete as affecting reinforcing metal.
(5) The erratic results obtained by the Vicat needle in ascertaining the initial set of cement.

The Reinforced Concrete Practice Standing Committee have drafted reports on the subject of "The Standardisation of Drawings for Reinforced Concrete Work" and "The Consistency of Concrete," which have been discussed at ordinary general meetings of the Institute. The Committee have also had under consideration a reference of the Council for a report as to clauses in contracts for reinforced concrete work, but the Committee recommended the Council that they were of opinion that the formulation of clauses in contracts was a legal matter, and the Committee did not feel they were capable of formulating such clauses. The Committee are still investigating:—

(1) Methods of treating the surface of concrete.
(2) Cracks due to the expansion and contraction of reinforced concrete.

APPENDIX.

The following is an Abstract of Report of the Committee appointed to consider the widened scope of the Institute, and which was presented as an Appendix to the Annual Report:—

On October 26th, 1911, a proposal for extending the scope of the Institute was considered by the Council.

After considerable discussion it was unanimously resolved that:—

"The Council do approve in principle the proposal for enlarging the scope of the Institute."

It was also unanimously resolved that a Committee be appointed:—

(*a*) To consider how the above resolution could best be carried out.

(*b*) To take energetic steps to develop the Structural Engineering side of the widened scope.

The Committee of ten members of the Council was elected, and Mr. E. Fiander Etchells was appointed Chairman.

At a series of meetings it was resolved that:—

Scope.—(*a*) Clause 3 (1) of the present rules does not limit the scope of the Institute to concrete and reinforced concrete, but that the clause enables the Institute to deal with iron (including steel), bricks, gravel, sand, cements, and other structural materials and their application.

Definitions.—(*b*) For the purposes of the Institute, *Structural Engineering* be defined as that branch of Engineering which deals with the scientific design, the construction, and the erection of structures of all kinds in any material.

(*c*) *Structures* may be defined as those constructions which are subject principally to the laws of *Statics*, as opposed to those constructions which are subject principally to the laws of *Dynamics* and *Kinematics*, such as engines and machines.

Title.—(*d*) After very long discussion, the Committee came to the unanimous decision that the term "The Concrete Institute" should remain in the title, and that it should have the first place in the title, and that a sub-title should be added, so that the full title and sub-title should be—

THE CONCRETE INSTITUTE,
An Institution for Structural Engineers, Architects, etc.—

and that such description should be added to all documents wherever practicable.

Lectures and Examinations.—(*e*) An annual course of technical lectures on some branch of Structural Engineering be instituted.

(*f*) Examinations in Structural Engineering be held by the Institute once a year, to test the scientific or technical attainments of applicants for studentship.

Subscriptions.—(*g*) An entrance fee of one guinea be required when the membership reaches one thousand.

"*Transactions.*"—(*h*) "Transactions" should be issued quarterly.

(*i*) Members should be requested to return the reports of discussions at meetings not later than seven days after receipt and that the following circular letter be sent to speakers:—

> "In order to facilitate publication of the 'Transactions,' early return of the enclosed report of your contribution to the discussion of the above-mentioned paper is requested. The Council have directed that reports which are not returned to the Institute within one week shall be published in the form shown."

(*j*) Members be hereby invited to submit a note as to suggested papers covering all branches of structural engineering.

(*k*) New books received should be reviewed in the "Transactions."

(*l*) A referendum be employed to ascertain the wishes of the members as to:—

1. The most convenient meeting-place.
2. The most suitable hour of meeting.

(*m*) The time is not yet ripe for any change of rules, but that the suggestions in this report should be worked upon for twelve months as a trial.

The Committee desire to state that valuable information has been collected in connection with the revision of rules, and this information will be filed for future reference, and is to some extent embodied in the minutes of the Committee's proceedings.

DISCUSSION ON THE REPORT.

The Chairman (Sir Henry Tanner, C.B., F.R.I.B.A.), after moving the adoption of the Report, surveyed the present position of the Institute as to its membership and finance.

With regard to the London County Council Regulations that had occupied a lot of time of most of the members of the Councils and the Sub-Committees, he thought they had done very good service in what had been sent to the Local Government Board.

As to the scope of the Institute it might be thought that the Report of that Committee did not go very far; still, it had to be made quietly to see how things worked, and they hoped they were going in the right direction to make the Institute of more extended interest to people, and therefore to increase the membership.

Regarding the Library, they had got scarcely any books, but they wished as many members as possible would make gifts of books. There were probably many members who would be very anxious to do that if they only knew what was wanted.

The Science Standing Committee, the Tests Standing Committee, and the Reinforced Concrete Committee had all been to work, and the Report showed all that they had had to do during last session, and the Institute ought to be very thankful to these Committees for giving so much time and attention to the business.

The Lectures had been dropped this time, but it was one of the proposals of the Committee on the scope of the Institute that these should be commenced again next session.

Mr. Edwin O. Sachs, F R.S.Ed., V.P.C.I., in seconding the Report, said that it contained much that would be of interest to the majority of members who were far away, and he trusted they would read the report carefully.

He wished to call attention to the fact that they were losing their President, Sir Henry Tanner, and this, they would all agree, was a very great loss. Sir Henry Tanner had steered the Institute through two years of very critical times. To put it quite plainly, Sir Henry Tanner, he thought, had held the Institute together. When he started his presidency it was only about three years old. It was a very young child, but was somewhat of a handful. It seemed to have all the ailments that children are apt to get. But Sir Henry Tanner acted as nurse and doctor in one, and it grew to be robust, if not larger in size, under his treatment. Latterly the Institute had also made a fresh spurt at growing, and thus, when Sir Henry Tanner left the Chair, it would fall to our friend, Mr. Wells, to see that this was no mere spurt, but a regular and healthy growth.

It should be pointed out very plainly to what a very great extent they were indebted to Sir Henry Tanner for having guided them, and he would take the opportunity of emphasising that point at this meeting, as he understood any remarks on the Report appear in the "Transactions," and our many members who were away from here will thus be informed of the views of the meeting of the great services rendered by the retiring President.

He had very much pleasure in seconding the adoption of the Report.

Mr. Lucien Serraillier.—He did not think it desirable to ask the assistance of members in canvassing for advertisements in the "Transactions" and thought the paragraph regarding this should be left out of the report.

The Chairman asked if anybody seconded this proposition, and, after a pause, said there was no seconder.

Mr. H. Percy Boulnois, M.Inst.C.E., V.P.C.I., said he would like to support the adoption of this Report and to congratulate the Council upon the work which they had done. With regard to the proposed lectures, he took it that the lectures were paid for and that the students attending paid a fee. That was not stated in the Report, and young men might ask him, and he should not be able to tell them anything about it.

With regard to the "Transactions" being issued quarterly, that would, he thought, be very expensive, because he noticed now that the "Transactions" and other printing had cost £228, and unless they got the advertisements they would make a very heavy loss.

The work that this Council had been doing was extraordinary in the way of investigation, and the list of things they had either considered or were about to consider almost appalled him, because he had been elected on the Council, and if he were to take part in this gigantic work he really trembled to think what the consequence might be. There was a list of seven matters which they were going to consider, from reinforced concrete piles down to the effect of oils and fats on concrete. On page 9 they had got the effect upon steel of the presence of sulphur in aggregates, and four other matters, winding up with the erratic results obtained by the Vicat needle in ascertaining the initial set of cement, and the methods of treating the surface of concrete and cracks due to expansion and contraction of reinforced concrete. All that was a terrific programme, and he only hoped they would be able to get through one-tenth of it.

He would like to suggest another investigation, and that was when they heard of failures in concrete that a Committee should thoroughly investigate the reason of those failures. Fortunately they had not heard of so many lately, and they were mostly exaggerated; but something should be done that when they did hear of any failure in reinforced concrete they should thoroughly investigate it and see if it was due to the reinforcement of the concrete, or to some other cause.

Mr. W. G. Kirkaldy, Assoc.M.Inst.C.E., in supporting the Report, suggested that in the "Transactions" the advertisements, if obtained, be not embodied in the pages of the ordinary matter, but kept at either end.

Mr. F. E. Wentworth-Sheilds. M.Inst.C.E.—He was very pleased to see that the Committee who had in charge the suggestions for widening the scope of the Institution had suggested as a sub-title "An Institution for Structural Engineers, Architects, etc." He was very delighted to see that "etc.," because there was a little tendency at one time for their Improvement Committee to rule out anybody excepting engineers and architects, and he thought it was the strength of this Institute that they had been able to combine so admirably the brains and the work and the good fellowship of so many different men, representing the building trade and profession in all sorts of different ways.

About advertisements, he sympathised with Mr. Serraillier's idea. It was not very nice to have to advertise, but, on the other hand, it seemed that they must face the problem in this way. They had a great deal of expense, and not enough money coming in for that expense. Well, surely it was better to ask for advertisements, and to put advertisements at one end or the other, or both, of their Journal, rather than raise the subscription and so make it more difficult for members who would like to join the Institute and get its advantages to come in and join their ranks.

The Chairman.—There were not many points which he had to answer. The principal one was the observations of Mr. Sachs, in which he used such flattering words in regard to himself. He was not aware that he had done so much as Mr. Sachs said. Of course, he had had the interest of the Institute at heart and had done what he could for it and to help everybody along; but then, probably everyone would have done the same. They all worked together, and it so happened that it had come round all right so far. However, he was very much obliged to Mr. Sachs.

Regarding the advertisements, they wanted these, and would put them at both ends of the book. The Institute of British Architects did it, and the Association also, so that there was no harm in their doing it also.

With regard to the question about the lectures, they had only one course of lectures, and they neither paid the lecturer nor did the students pay a fee. He did not see why they should not, and also why the lecturer should not have something for his trouble. Anybody who wanted lectures by way of instruction ought to pay something for them. That is a matter which the Council would have to consider before they issued the particulars.

As to the printing of the quarterly issue, they found last year there were a good many complaints as to the irregularity of the publishing of the "Transactions." They got rather out of date, and people forgot all about them, and they were not of so much service; so it was settled, under this new Committee, to issue them quarterly. The size had been cut down and the type altered. They were quite adequate for the purpose, and it was also proposed still further to reduce the expense by sub-editing some part of what appeared there in the future—that is to say, they were not going to put everything in full, but were going to leave out some of the particulars.

The subject referred to by Mr. Boulnois should be examined into—the failures, and so on. Of course, everybody wished to do that, but it was not so easy to do it. You cannot publish those things, sometimes, without interfering with people's feelings.

The Chairman then read the Report of the Scrutineers on the Council's election.

PRESENTATION OF MEDAL TO PROFESSOR BERESFORD PITE, F.R.I.B.A.

The Chairman.—The next business was to present the medal to Professor Beresford Pite; and in presenting that to him for his paper on "The Aesthetic Treatment of Concrete," which the Council had awarded, he would like to say that it was a most interesting contribution to the literature of the subject treated, and it gave rise to some little discussion, although that was very limited in extent. The architects did not turn up, as they had hoped, to take a considerable part in it.

He was glad that Professor Beresford Pite did not altogether object to what had been done and advocate something entirely new, because they should have got into a great deal of trouble. But he did not do that; he said they ought to start from the existing style of architecture, and that seemed really to be the most satisfactory way of meeting this difficulty, because it was a real difficulty. Nobody had developed anything yet which when new was satisfactory, or, perhaps, when it was old it was rather better, but still not what might be expected from the treatment of reinforced concrete. However, there was no doubt that something would develop shortly, and the Professor had given them a start in that direction. Perhaps the Royal Institute might develop a discussion itself, and so make some greater effort than had been done hitherto. He would ask the Professor's acceptance of this small memento of their regard for him and his lecture.

The Chairman then handed Professor Beresford Pite the Bronze Medal of the Institute for the best paper read in the 1910-1911 Session.

Professor Beresford Pite, F.R.I.B.A., after thanking the Council of the Institute most sincerely for the honour they had conferred upon him by bestowing this medal for the paper, said the subject of his paper was an aspect of work in concrete that had not yet captured the imagination of members of this Institute as he hoped it would in the future, as one could not but be sure that if the apparent fog that hung over progress in the architectural aspects of concrete construction were dispelled by a little clear light there ought to be a considerable acquisition of width and breadth to the operations of this Institute. So long as this subject remained unexplored and unexperimented with, so long would this Institute be looked upon as merely a body concerned with certain aspects of building, and concerned with certain aspects of building only, and concrete construction, especially in its more prominent methods now, would be relegated to the larger, coarser, and greater class of building, when it might, if there were interest and enthusiasm and hope and light, be used for more monumental, more decorative, and more homely, and therefore much wider purposes.

It seemed to him that since it was recognised that there was a field for experiment, a field for movement, a field for intellectual thought, a field which connected a decent contract in concrete with an ordinary and regular course of architectural study and æsthetic consideration, then there would be the widened scope which their Special Committee so hoped for and which they had in a way suggested by adding the word "Architects" to the title of the Institute. That widened scope would come, and a very broad and very large, and he would like to say very popular, appreciation of the possibilities of concrete, because the popular aspect of this subject was, of course, one closely allied to the commercial one. The commercial aspect and the popular aspect ought to make this Institute a very wide and a very important public body.

He would only like to make one suggestion with regard to this, which was a practical suggestion, and one not at all inappropriate, connected with the honour that they had ventured most courageously to bestow upon an unworthy person to-day, and that suggestion was that this subject should not be dropped, that the attention of architectural students and designers should be drawn to it. The Chairman had indicated that the Institute of British Architects did not pay much attention to it, and he was perfectly right. If he had said bluntly that they were prejudiced with regard to the whole subject of concrete he would only have spoken what he felt and what with some delicacy we know he hesitated to express. The best way to disperse that prejudice would be to show what great and good designs could be perhaps imagined, because every castle had to be built in the air before it could be built on the earth by young and enterprising designers, and the institution by this Institute of an annual competition in designs would, he thought, attract, at all events, a certain number of responses; but he was sure it would attract a few intellectual and careful thinkers on the subject of æsthetics. Those students who had little else to think about in their youth but the charm of design would turn with interest to such a competition; and if the medal which they had now created and which was such an artistic object were bestowed with such monetary attraction as either a special fund or some contribution from the Institute as it might be able to afford annually for a design, the subject of any design to deal with the simply architectural aspects, elevation, and details suitable for application to or distribution in reinforced concrete with, if necessary, any descriptive theory as to the method of design, he thought, without expecting too much in the way of hard labour from the competitor, the Institute would be put into possession annually of a large number of interesting artistic essays—*i.e.*, artistic inventions, artistic designs, and would attract to itself the attention of students; attract them to its meetings and, having got them, no doubt they would be as beneficial to the publication as the advertisements with their illustrations at the end.

Personally, he always looked upon those advertisements with a great deal of interest when they dealt with technical and practical subjects. The illustrations of deviations in reinforced concrete design would have a most important influence upon the whole subject. Anyone who took up the "Transactions" will say that was a live aspect of the question; they would attract those of the general public who were very widely interested in the outward aspects of the buildings, and they would also attract the attention of art critics and others who certainly should be made conscious of the fact that a new and effective building material had come into practical use, and the world would move, as it always had moved in these artistic matters, slowly but with interest and life and with enthusiasm.

NEW PRESIDENT.

The Chairman.—The next business was for him to vacate this chair and to put Mr. Wells into it. In Mr. Wells he was sure they would have a President quite suitable to the Institute, not an amateur—one who was thoroughly acquainted with the business or with the profession of a designer in reinforced concrete, and he was sure Mr. Wells would make them a most excellent President, and he felt great confidence that the Institute would certainly advance under his guidance. He, therefore, would make way for Mr. Wells and wish him the best possible luck during his two years of office.

Mr. E. P. Wells, J.P., accordingly took the chair, and said he wished to thank Sir Henry Tanner for the very kind words that he had said, and for the manner in which they had received the same. He did not intend to make any remarks, but to say that during his term of office as President he trusted that in all his dealings with every member he should be thoroughly impartial, and he would do his best for the interests of the Institute.

VOTE OF THANKS TO SIR HENRY TANNER.

Mr. A. Alban H. Scott, M.S.A., proposed a very hearty vote of thanks to Sir Henry Tanner for taking the President's position for the last two years. Mr. Sachs had already practically said everything which should be said on an occasion like this. They would never get another President to take up such energetic work on behalf of this Institute as Sir Henry Tanner. He had given up an immense amount of time, and the courtesy which he had shown to every one with whom he had come into contact would also never be exceeded. He was sure that Mr. Wells would not take it that he would not give up as much time in one way, but Sir Henry had always been on the spot when he had been required, and if Mr. Wells was so fortunate as to be able to do that it would be rather a good thing. He had very great pleasure in proposing a most hearty vote of thanks to Sir Henry Tanner.

Mr. T. C. Dawson, in seconding Mr. Scott's proposal, said that the Institute and all the members were very much indebted to the labours of Sir Henry Tanner, who had thrown his whole heart into it during the last two years. As Mr. Sachs has said, he had steered them through rather critical times and worked exceedingly hard. There were not many institutions so fortunate as to have Presidents who had given so much energy to their work. He was only too glad to know that they would have his hearty co-operation afterwards, although not in the chair.

The Chairman (Mr. E. P. Wells) then put it to the meeting that a most hearty vote of thanks be accorded to Sir Henry Tanner for occupying the chair for the last two years as President. He would put that to the meeting, and he was sure everyone would join in it with acclamation. (Applause.)

The resolution was unanimously adopted.

Sir Henry Tanner, in replying, said he had great pleasure in listening to what they had said, and he thanked them extremely for the vote of thanks which they had given him for what he had done during the past two years. He could only say that it had given him very great pleasure to attend these meetings, and the meetings of the Council. He had made many friends there, and he hoped to continue to do so in a similar way. He would not care to continue as Chairman of any institute for very long; it was not quite suitable to him, and there were many who were much more gifted in the way of being Chairman and President. He thought they had got one for the purpose better than he had been hitherto. At the same time they had got their business through, and had arrived at a point which was certainly not backward from what it was two years ago, and for that he had to thank the members of the Council and the Secretary and all concerned.

The meeting then terminated.

THE SECOND ANNUAL DINNER.

The Second Annual Dinner of the Concrete Institute took place on May 9th, in the Empire Hall at the Trocadero Restaurant, Piccadilly Circus, W., Mr. E. P. Wells, J.P., President, in the chair.

Among those present were Mr. W. E. Riley, R.B.A., F.R.I.B.A., M.Inst.C.E., Superintending Architect of Metropolitan Buildings and Architect of the London County Council; Mr. R. Elliott-Cooper, M.Inst.C.E., President-Elect Institution of Civil Engineers; Mr. H. Percy Boulnois, M.Inst.C.E., V.P. Concrete Institute, Chairman of the Council of the Royal Sanitary Institute; Mr. E. A. Stickland, President of the Institution of Municipal Engineers, Borough Surveyor, Windsor; Mr. John Murray, F.R.I.B.A., Crown Surveyor; Sir W. Alfred Gelder, M.P., F.R.I.B.A.; Sir Henry Tanner, C.B., I.S.O., F.R.I.B.A., Past President of the Concrete Institute, Principal Architect to H.M. Office of Works; Mr. F. E. Wentworth Sheilds, M.Inst.C.E., V.P. Concrete Institute, Dock Engineer of the L. & S.W. Railway; Captain A. Boyd Carpenter, Ex-Mayor of Harrogate; Professor J. H. Cormack, B.Sc., Assoc.M.Inst.C.E., Professor of Mechanical Engineering, University College, London; Mr. H. H. Dalrymple Hay, M.Inst.C.E., Chief Construction Engineer Underground Railways, London; Mr. C. R. S. Kirkpatrick, M.Inst.C.E., Chief Assistant Engineer, Port of London; Mr. W. J. Downer, I.S.O., J.P., Assistant Secretary of H.M. Office of Works; Lt.-Colonel G. E. Holman.

Mr. W. E. Riley, in proposing the toast of "The Concrete Institute," congratulated the Institute upon the great success which had been achieved already. Pioneer work was generally a very thankless task. He suggested that those who had inaugurated the Institute had very far-reaching thought because whether they liked it or not concrete and reinforced concrete generally had come to stay. Those who had been through an architectural training understood the problems he meant, but he believed that the man who was going to make a mark in life was the man who would show adaptability and commercial interest. The Institute was the absolute embodiment of that idea. He had heard men say they were sorry that someone had discovered that the putting of steel down concrete made it strong; the commercial world was not sorry for that, it was going to be glad. He had always felt it his duty to make rigid laws elastic by administering them with benevolence, and he reminded those present that the aims of concrete experts had been greatly aided by three or four changes in the law which, though based on rigid principles, would be found to be more elastic in application than might be thought, and which encouraged the application of new materials. He had recently been at the Architectural Congress at Rome, and he was glad to see that a good deal of time and consideration was given to the question of the design of reinforced concrete, though the conclusion there was no one knew how to treat it. Some thought it papery, thin and expressionless, etc., but the right thing would evolve. He thought the aims of the Concrete Institute would result in obtaining proper and artistic expression by those who were trying to interpret building materials.

He felt somewhat diffident in speaking to an Institute like this about regulations, but the onerous duty had been thrown upon him of trying to do something to put the regulations upon a proper footing. Nothing would be ideal, but the regulations should be made as ideal as could be. Any amount of personal effort had been put in to try and make them so. It was a very cheap thing to say that he had been saying two and two made four; he knew he had said that before on several occasions, but he had always had to make two and two make four, although some people tried to make it five. There was something to be said about the regulations, and that was the only opportunity he had had of saying it. Those in authority at the London County Council were deeply grateful for the enormous assistance which they had had from this Institute in endeavouring to pioneer and do something in London. Of course, he was not going to say anything more about the regulations, because they were *sub judice*, but he hoped that they would soon appear and that they should know where they were. They went through every possible source of information they could think of and tried to get the practice of other countries to codify them for the benefit of all.

He congratulated the Institute on having an eighteen-carat man as the out-going President. He had known Sir Henry Tanner both officially and in private life, and the present President, Mr. E. P. Wells, was an old friend and an old opponent; he was an old colleague of his at the London County Council and was appreciated and respected there. He thought they had done the right thing in making Mr. Wells President of the Institute. He could not help saying this about the long-looked-for regulations, that they aim at trying to help all thoroughly and to do what was wanted—namely, to give a filip and to give the benefit of their enormous thought, experiences, and services to those who were willing and anxious to give commercial development to the concrete work which they had been endeavouring to bring to fruition. He was glad to have opportunity of thanking the Institute for the great help they had given him personally in endeavouring to make those regulations such that they would be workable and practicable.

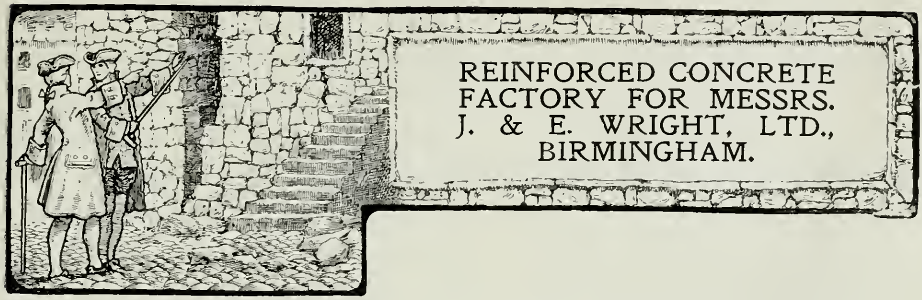
Mr. E. P. Wells, J.P., President of the Concrete Institute, in thanking Mr. Riley for the kind words he had said on behalf of the Institute and about himself, said he also thanked him for the extremely impartial attitude that he had taken all through the negotiations that had passed between him and the Concrete Institute in regard to the new regulations about to be laid down for reinforced concrete, and also for the fact that everything that had been done had been carried out in the best of temper. He was happy to say that in every case where suggestions had been made by a member of the Institute they had been listened to, and where they had been perfectly just they had been accepted without a murmur. He trusted that in the negotiations which were about to be carried on by the Local Government Board they would be carried on in the same spirit as existed between the Concrete Institute and Mr. Riley. The regulations put forward by the London County Council had for their primary object the protection of life and property. Personally, he would uphold those regulations throughout because he felt convinced that, though they might at the time appear a little hard, still they were for the benefit of London as a whole. If buildings can be put up which can be made absolutely fireproof, though it would be more costly, still it would be cheapest in the long run; therefore he was of the opinion from what he had seen and what he had had to do with the Building Acts that they might be made more strenuous than at the present time. He hoped that if anything was done in the future for their amendment that certain clauses would be added so as to place everything in such a condition that no one would be able to alter in any shape or form, and whenever they had a building in accordance with the regulations they might be perfectly certain that if a fire took place very little damage would be done, if any, outside the building where the fire originated. There might be one or two present who thought that the regulations were a little hard, but he was of opinion that when the Act was thoroughly gone into and applied it would be found that they would be beneficial all round.

Next session the Council of the Concrete Institute were going to institute educational lectures for the younger men, and he trusted that they would be attended not only by the younger but also by some of the older ones, so as to give every help to the lecturer. He was happy to say that the total number of members was 920, but hoped before the end of the year was out that the number would be 1,000, and that in the next year or so they might see the number to 2,000.

Mr. F. E. Wentworth-Sheilds, Vice-President, Concrete Institute, in proposing the visitors, said they were able to point with pardonable pride to their membership, which, although the Society was so young, was now getting nearly a thousand strong, and they were able to say that the work which they had done had been useful not only to structural engineers, but to the world at large, and particularly to the citizens of London. They were able to say they had formed rules by which the regulations and the drawing of reinforced concrete designs would in future be simplified. They were glad to point to the fact that they had been able to draw up rules under which they hoped that in future their structures would not only be strong, but would last for their generation and many generations to come. They were able also to say with satisfaction that Parliament had entrusted them with the work of offering their criticisms and their suggestions to the rules which the London County Council had drawn up for the guidance of builders in reinforced concrete. Of the kind things that Mr. Riley had said to the gathering about some of their criticisms, he had not said that they were beyond the mark, or if he did he certainly did not say so above a whisper. He thought they might say that not only had they come to feel pleasure but also pride in the little work they had been able to do, but they could also derive gratification that their guests there that night had, by their presence and acceptance of the hospitality, expressed their pleasure at the useful work done not only to the profession, but to the world at large. There were so many distinguished guests that it would take too long to mention them by name, but he would like to say how pleased they were to see the President-elect of the Institute of Civil Engineers. It would be recollected that last year they had the honour of entertaining Mr. Siemens, who was then President, when he reminded them of the fact, of which they were fully conscious, that that august body was the mother of all engineering societies, indeed he believed he said the grandmother. (Laughter.) He should also like to say how pleased they were to see Mr. Riley and other distinguished guests, Presidents of learned societies, and so on.

Lieut.-Colonel Holman, in responding, said he had known the President of the Concrete Institute, Mr. E. P. Wells, for many years, and the longer he had known him the more he had learned to respect, admire, and love him. The most scientific of men, he was ever anxious to do things on the right lines. He was now clamouring that the regulations should be made more stringent. He did not hear Sir Alfred Gelder say anything to that, and he did not hear architects clamouring for the same thing. They were not clamouring for more strenuous regulations, but Mr. Wells made him anxious because architects must consult engineers from the Concrete Institute. But he was sure of this, that somebody who knew Mr. Riley would endorse all he had said about the administration of the Acts, and that they would be administered with benevolence and not that rigid, cast-iron rule as outside the London area. Architects had no complaints to make of the administration of the law.

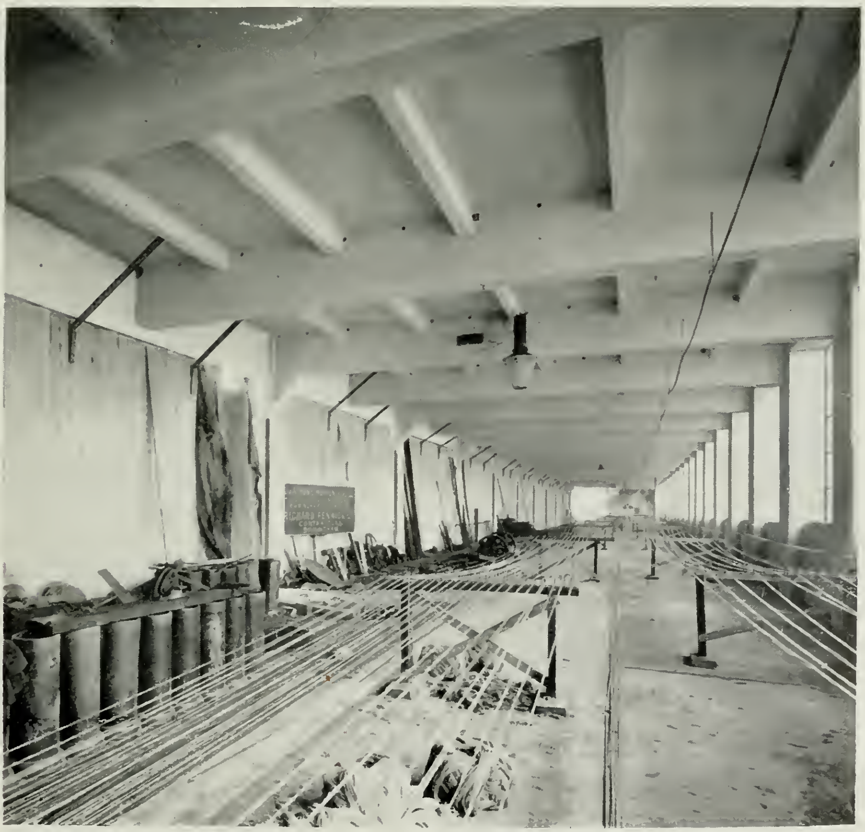
Sir Henry Tanner, F.R.I.B.A., proposed the toast of the President, Mr. E. P. Wells, which was drunk with musical honours, and Mr. Wells briefly responded.



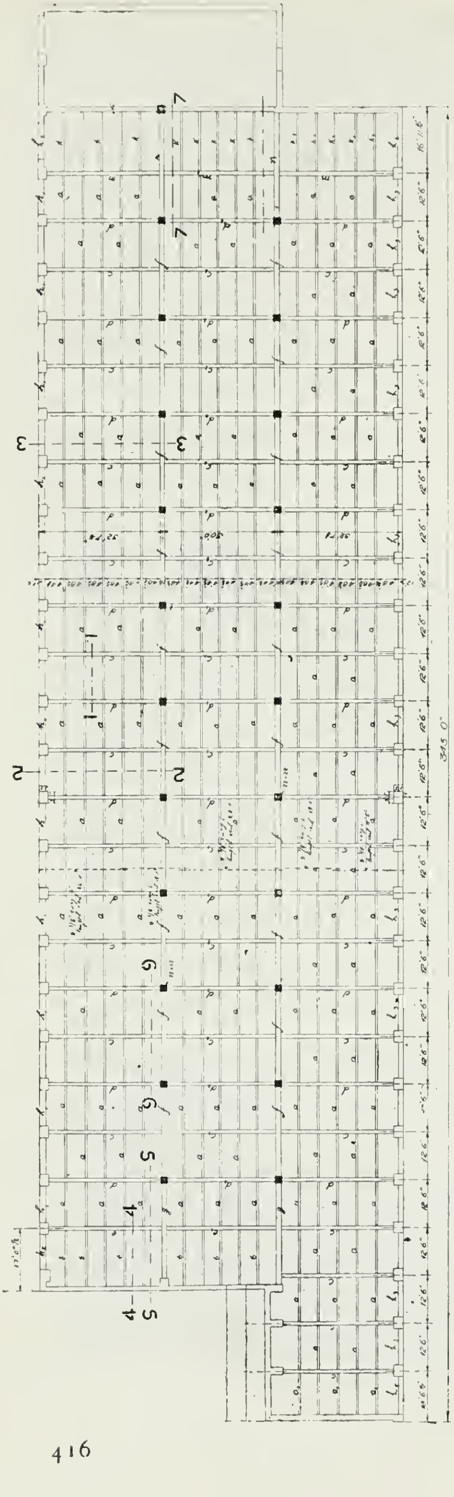
REINFORCED CONCRETE FACTORY FOR MESSRS. J. & E. WRIGHT, LTD., BIRMINGHAM.

The following account of a large factory in course of erection at Birmingham should be of interest. It is constructed of reinforced concrete throughout with the exception of the walls, which are in brick, and is a good example of the suitability of concrete for factory buildings.—ED.

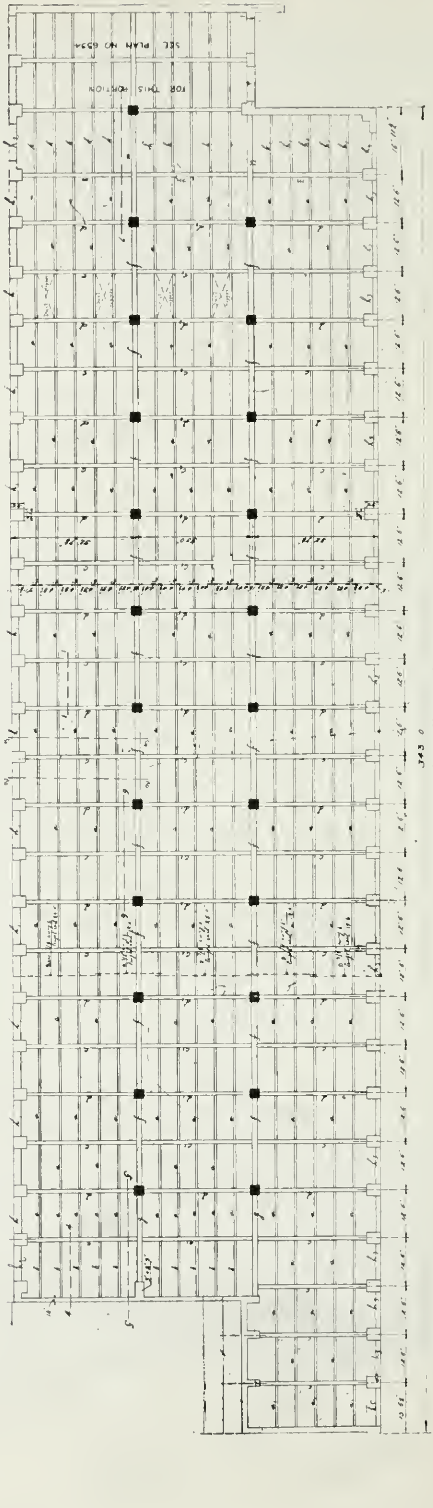
The illustrations and plans reproduced in this article are those of a large building in course of construction for Messrs. J. & E. Wright, Ltd., at Universe Works, Garrison Street, Birmingham. Messrs. Wright are the old-established



First bay completed ready to receive the machinery.
Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.



Second Floor Plan.



First Floor Plan.

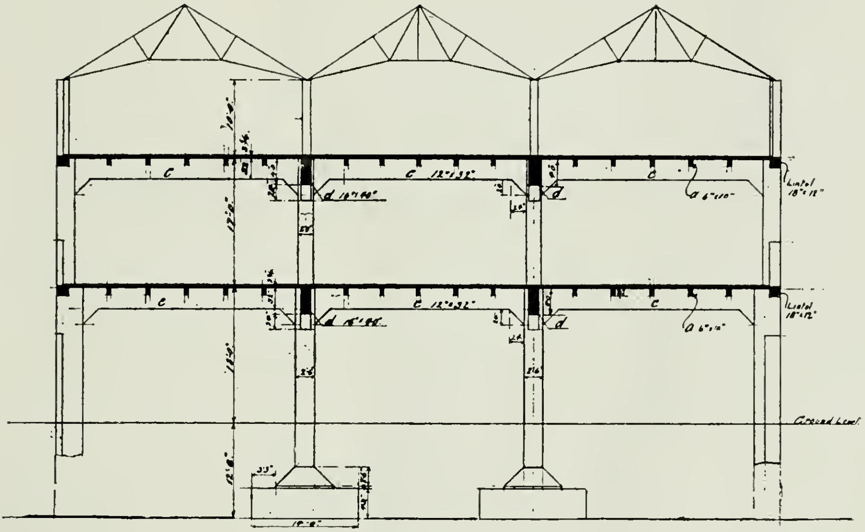
Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.

firm of rope manufacturers, whose specialities include steel wire ropes for mining purposes, cotton driving ropes, and hemp ropes of every description.

This large building, which is to form an extension to Messrs. Wright's important works, has been designed to contain machinery.

The total length of the front elevation of the complete building is 343 ft., divided up into bays of 12 ft. 6 in. centre to centre of the pillars. The width of the building is approximately 95 ft., divided up into three bays in such a manner as to provide for a double row of reinforced concrete pillars throughout the centre of the building.

As shown in the cross-section, the work comprises two reinforced concrete floors and a steel roof. The total area of each of the reinforced concrete floors is approximately 3,766 square yards, not including the annex or store and the lavatories on the three floors, the total area of which is approximately 234 square yards.



Cross Section.

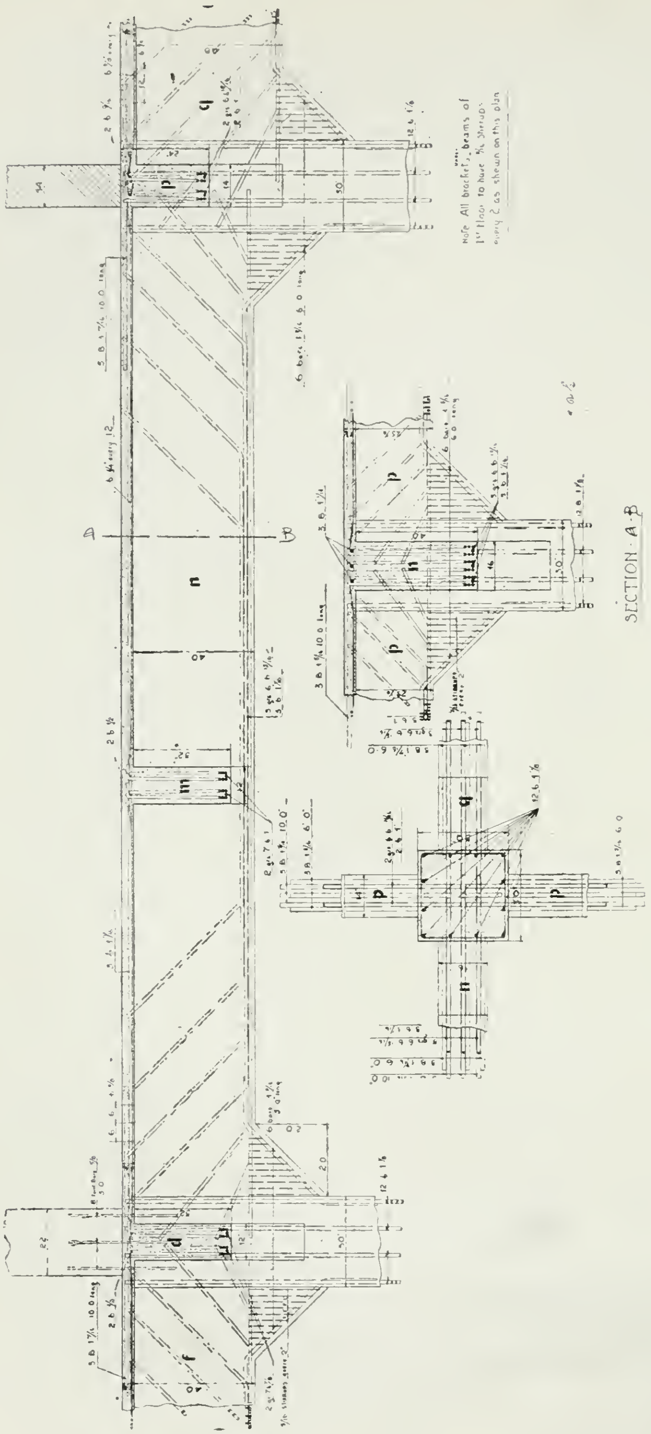
Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.

The first and second floors have been calculated for a superload of 3 cwt. per sq. ft. The store floor, however, has been calculated for a distributed load of 10 cwt. per sq. ft.

In addition to the above-mentioned superloads, some of the pillars and beams have been designed to support crane roads.

Although practically the entire structure is in reinforced concrete, the engineers have found it desirable to meet the Local Building Regulations by constructing the walls in brickwork from 1 ft. 11 in. to 14 in. in thickness, with substantial piers to support the ends of the main beam on first and second floors.

A glance at the accompanying plans shows the method of arrangement of the pillars, principal beams and secondary beams. It will be seen that the


SECTION · A-B
Note All brackets, beams of 1st floor to have ⅜ stirrups every 2 as shewn on this plan
A
B
n
m
p
q
f

Detail Section of a Post.

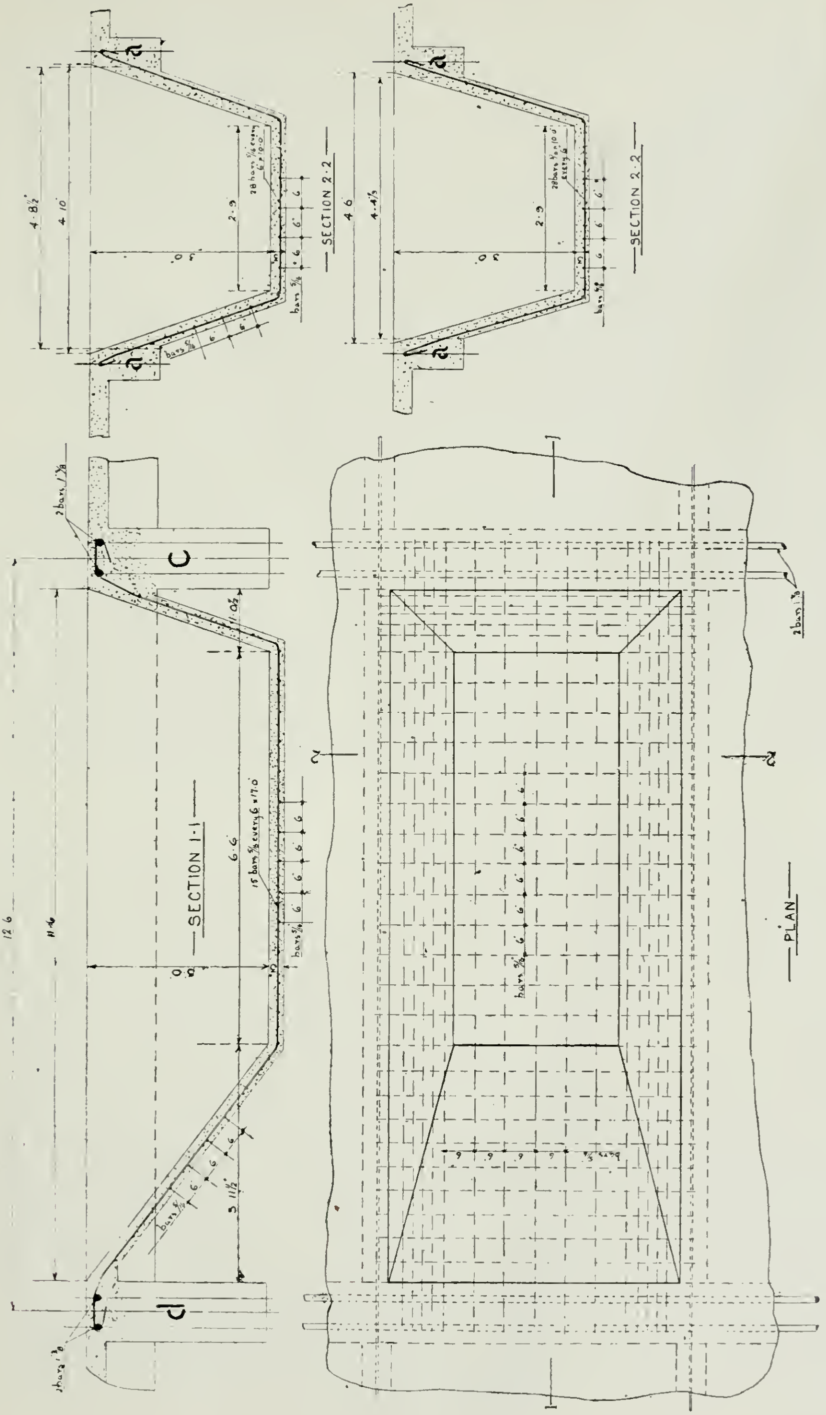
Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.

pillars on the ground floor measure 30 in. by 30 in., and those on the first floor 22 in. by 22 in. The roof over the second floor, however, is supported by steel stanchions, as the latter will form a better connection to the steel roof.

The main beams on the first floor running in a longitudinal direction between the reinforced concrete posts have scantlings of 16 in. by 40 in. The principal beams running at right angles to these have scantlings of 12 in. by 32 in. The latter support five secondary beams of 6 in. by 10 in.

The thickness of the reinforced concrete slab is $3\frac{3}{4}$ in.

The scantlings of the structural members of the second floor are



Plan and Section of Dust Hoppers.

Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.

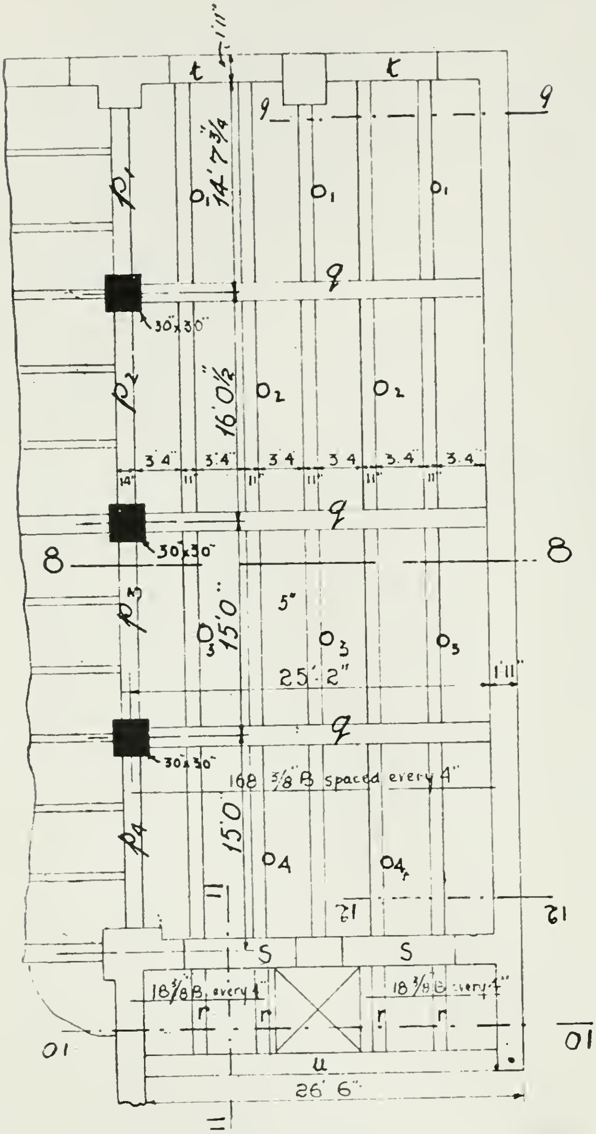
identical with those of the first floor as they have been calculated for the same superload. The pillars have been provided with reinforced concrete footings in order to distribute the load on to blocks in ordinary concrete, calculated to spread on the ground a load of about 2 tons per sq. ft.

In certain parts of the floors the slab between the secondary beams has been recessed to a depth of 3 ft. in order to form dust hoppers.

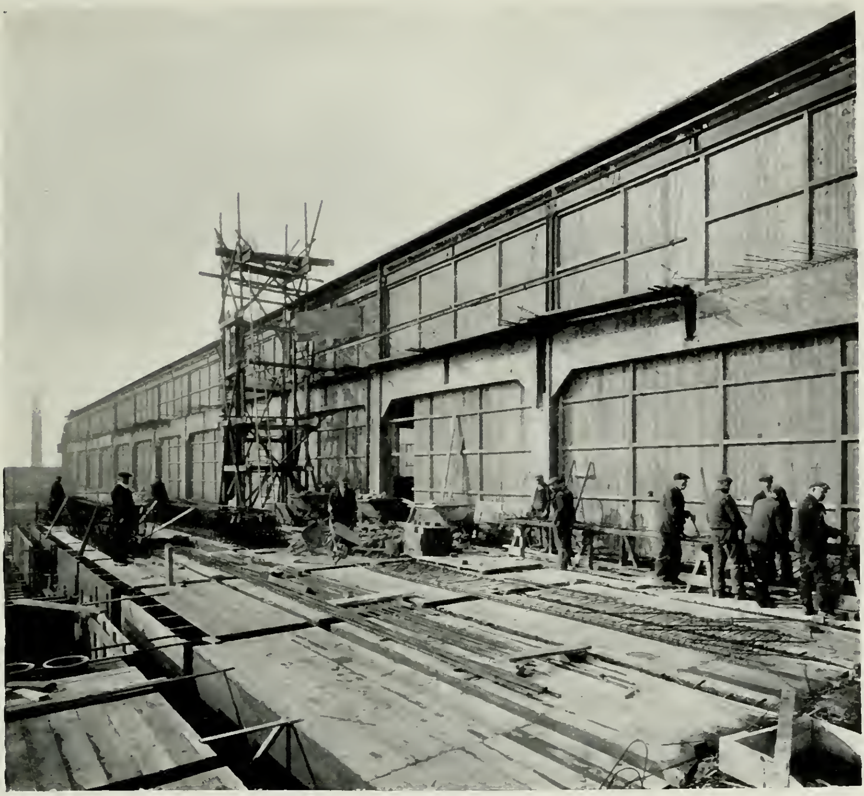
Our illustration on page 415 shows the first completed bay ready to receive the machinery. The holes shown in the main beams have been provided with a passage for pipes, cables, bolts, etc.

The illustration on page 421 shows the second bay during the course of construction. The photo was taken during the preparation of the unit reinforcement for the main beams.

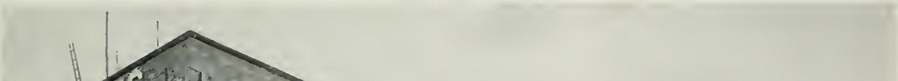
The reinforcement, which is on the Coignet System, is composed entirely of round bars arranged in the manner shown on the detail section which is reproduced on page 418. It will be seen that for the main beams three groups of bars have been provided to resist the tension. Each group comprises 7 bars, 6 of which have their ends gradually bent upwards at an angle of 45°, the extremity of these ends being bent in the shape of a hook and fixed over a top longitudinal bar in such a manner as to prevent any slipping of the bars through the concrete.



Plan of Store Floor.
Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.



View of Second Bay in Course of Construction.



View of Elevation facing Railway Line.
Reinforced Concrete Factory for Messrs. J. & E. Wright, Ltd., Birmingham.

One of the advantages offered by this method is that the section of steel in tension gradually decreases towards the support in proportion to the curve of bending moments, the ends of the bars being bent upwards to resist diagonal tension or sheer.

A certain number of additional vertical stirrups are also provided in order to completely absorb the sheering efforts.

The reinforcement of the pillars is composed of 12 bars bound together by ties of small diameter in order to keep the main bars in their proper position during the concreting.

The engineers for the work are Messrs. D. & A. Home-Morton, MM.Inst.C.E., consulting engineers and industrial works designers, of Glasgow and Birmingham.

The reinforced concrete, which is entirely on the Coignet System, is being constructed by Messrs. Richard Fenwick, Ltd., contractors, of Birmingham.

A NEW APPLICATION OF CAST-IRON IN COLUMNS AND ARCHED BRIDGES.

By DR. F. VON EMPERGER.

The particulars given below are taken from a very interesting article by Dr. Von Emperger which appeared in our contemporary, "Beton u. Eisen." This article has recently been issued in pamphlet form by Messrs. Wilhelm Ernst & Sohn, of Berlin, at the price of M2.50. The German title is "Eine Neue Verwendung des Gusseisens bei Saulen und Bogenbrucken."

Reinforced concrete columns, in which the reinforcement does not exceed 2 per cent., do not owe any appreciable part of their load-carrying capacity to the steel, which only strengthens by resisting shear. In the system of construction now described the metal and concrete combine to carry the load.

Cast-iron Columns.—The use of cast-iron columns in building has greatly declined in recent years. Their defects can, however, be removed by a suitable combination with concrete. The experiments now to be described show that the dangerous brittleness of the material is neutralised, and a structure is obtained which is capable of undergoing great deformation without fracture. It is claimed that the resulting members are the strongest and cheapest of all forms of columns. The twenty-four experiments were carried out by the Austrian Reinforced Concrete Committee.

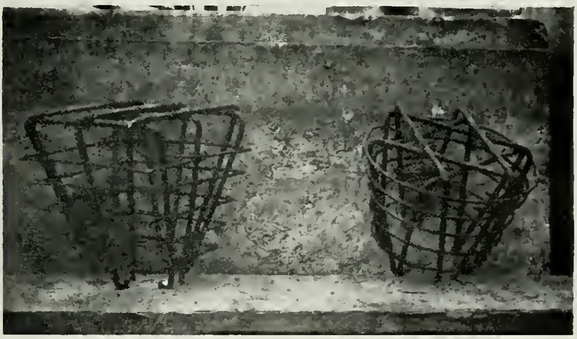


Fig. 1. Formation of Column Heads.

Experiments with Cast-iron Pipes.—Four similar iron columns were used, one being tested by itself and the three others with different coatings of concrete, the age in each case being 61 days. The unprotected column (*Fig.* 2), with a section of 40·1 sq. cm. (6·2 sq. in.), broke under a load of 137 tons, which amounted to 3,420 kg./cm.^{2}, fracture taking place so suddenly that the testing machine was injured considerably. This load was in accordance with the usual crushing strength of cast-iron, less the correction made for bending of a long column. Column II. (*Fig.* 3) was spirally wound with 10 mm. wire (0·4 in.) at intervals of 4 cm. (1·57 in.), with eight vertical rods, 5 mm. diameter (0·2 in.). Column IV. was similar, except that the vertical rods were replaced by No. 4 expanded metal. The thickness of the concrete coating was 63 mm. (2½ in.) to the outside of the reinforcement and 73 mm. (2·87 in.) in all, the outside 10 mm. being neglected in the statical computation. Capitals were formed as in *Fig.* 1. These two columns broke under loads of 315 and 307 tons respectively. The compressive strength of the concrete in the columns may be calculated to be 425 kg./cm.² (6,040 lb. per sq. in.), whilst cubes of the same concrete gave 352 kg./cm.² (5,006 lb. per sq. in.). In comparing the columns, however, allowance must be made for the greater tendency to bending in the first column on account of its slenderness.

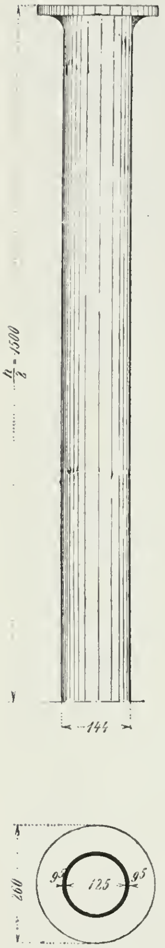


Fig. 2. Column I.

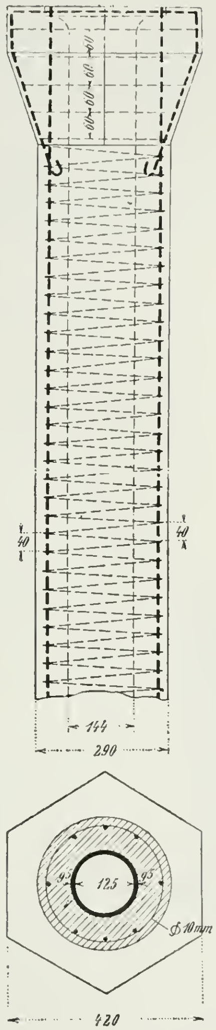


Fig. 3. Column II.

Column III. (*Fig.* 4) was similar, but with a closer spiral winding of 7 mm. wire. The breaking load was 342 tons. The fractured column is seen in *Fig.* 5, and it is evident that the dangerous brittleness of the cast-iron, causing fracture to take place almost explosively, has completely disappeared in the protected columns.

Safety of Construction.—Plain concrete columns are impossible in ordinary domestic construction, owing to the serious results that may be produced by small defects. Reinforced columns, with their protection against shearing, are safer; but certain serious accidents have shown that defects of construction may occur unless the reinforcement is of such a character as to be itself capable of carrying a load. This is costly when special rolled sections have to be used, but is cheap in the mode of construction just described. As moulds, two sheet-iron cylinders are used, and are placed alternately one above the other, so that special framing is entirely unnecessary.

Experiments with Mild Steel Tubes.—Of the eleven tests made, the first was with a solid concrete cylinder and the second a hollow cylinder, both without reinforcement. In both cases the concrete broke under 230 kg./cm.² (3,281 lb. per sq. in.). The third and fourth, solid and hollow cylinders respectively, were spirally reinforced, and gave increases of strength of 50 and 90 per cent. respectively. A mild steel tube was then tested alone in compression, as well as one which had been enclosed in concrete and the column crushed. It was found that the steel had not been appreciably weakened, which shows that in columns of this kind the strength of the steel is not available beyond the yield-point; whilst in the case of cast-iron the full compressive strength may be utilised. The following results were obtained when external reinforcement was also used:—

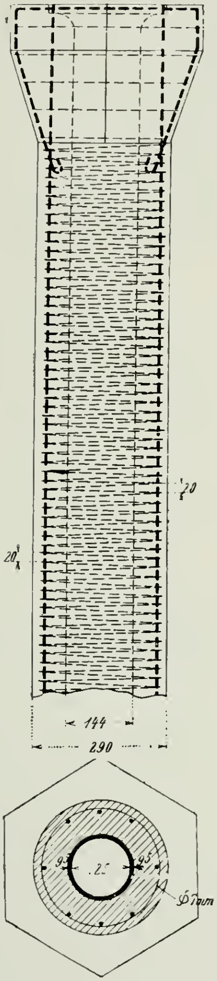


Fig. 4. Column III.

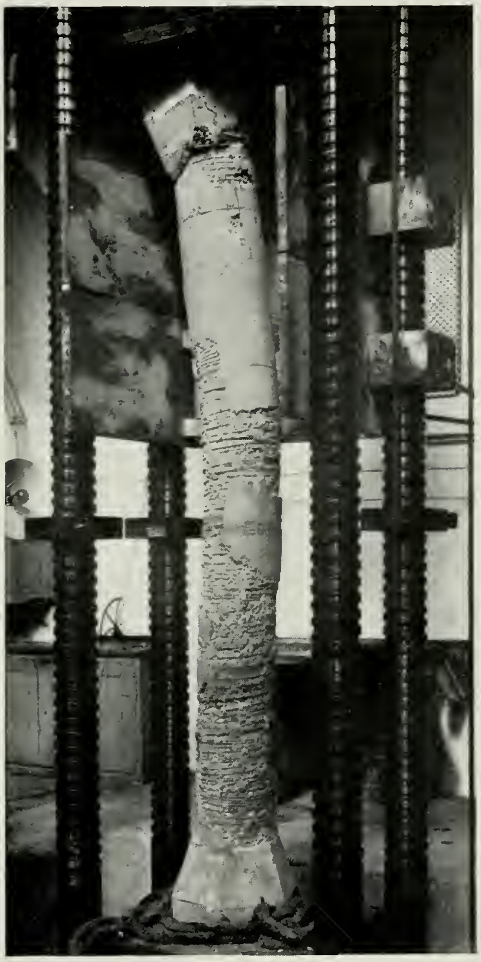


Fig. 5. Column III. under Load of 342 Tons.

Column	Vertical Reinforcement	Spiral Reinforcement		Load
VII.	8 rods, 5 mm.	10 mm.,	40 mm. interval	184 tons
VIII.	8 ,, 5 ,,	7 ,,	20 ,, ,,	205·5 ,,
IX.	8 ,, 5 ,,	10 ,,	40 ,, ,,	209 ,,
X.	8 ,, 5 ,,	Closed rings,	40 ,, ,,	198 ,,
XI.	Expanded metal	10 mm.,	40 ,, ,,	205·5 ,,

Cast-iron Columns.—Second Series.—These experiments were conducted with concrete which gave, in cubes, a strength of 187 kg./cm.^{2} (2,665 lb. per sq. in.) after 50 to 66 days. This agrees well with the strength determined from the experiments with columns. A hollow concrete cylinder without reinforcement broke under a similar load, whilst one reinforced with rods and hoops (*Fig.* 6) broke under a load of 45·7 tons, corresponding with 96 kg./cm.^{2} (1,366 lb. per sq. in.). The weakening is evidently due to the destruction of the outer shell by the vertical rods, so that only the inner part is actually carrying load, whilst the hoops are too widely spaced to be of use. Even reckoning only the inner cylinder as effective, the concrete is somewhat weakened, as the shock occurring when the outer shell bursts off affects the whole. The next column (*Fig.* 7), with spiral reinforcement, stood a load of 94 tons, or 179 kg./cm.^{2} in the concrete. A fourth column, with vertical rods and widely spaced hoops, confirmed the conclusion that this form of reinforcement weakens the concrete instead of strengthening it.

The remaining experiments of this series were directed to determining the proper form of casing for the cast-iron pipes. The author has laid down the principle that the spacing of the hoops must not exceed the thickness of the inner layer of concrete, between the core and the reinforcement. The tests show that when this condition is not fufilled the required strength is not attained. A factor of safety of 10 is required in the case of cast-iron columns, on account of the great brittleness of the material, and this disadvantage is not removed when the concrete casing is insufficiently reinforced, as shown by *Fig.* 8.

A cast-iron tube, 153 mm. (6 in.) diameter and 5 mm. (0·2 in.) thick, withstood a load of 209 tons (the length being 50 cm., or 20 in.); whilst the column shown in the figure, reinforced as shown in *Fig.* 9, only withstood 143 tons. One correctly reinforced with a close spiral. as in *Fig.* 10, only broke under a load of 332 tons. Moreover, the manner of fracture in the first case was similar to that of an unprotected tube, except that the violence was somewhat mitigated. The figures given for these columns are not directly comparable, as the iron tubes used were of slightly different size; but a calculation of the actual stresses shows that the badly reinforced column of *Figs.* 8 and 9 is actually 50 per cent. weaker than the unprotected iron. This is explained by the fact that, when the stress reaches a point at which the elastic limit of the vertical steel rods is exceeded, the load is suddenly transferred to the inner ring of concrete at the same moment that the outer shell is split off. The elastic limit of the concrete is then exceeded, and the load is, quite suddenly, transferred to the cast-iron.

Conclusions.—A comparison of a spirally reinforced solid concrete column with one of the same external diameter, with a cast-iron central pipe, shows an advantage of 25 to 50 per cent. in favour of the latter. It is therefore possible to construct comparatively slender columns, which was formerly only the case by the use of cast-iron, and only one-half to one-third of the cross-section of iron is required. The cost is small, especially when allowance is made for the layer of concrete required in any case as a protection against fire. The quantity of steel required is evidently much less than in the construction of an ordinary spirally reinforced concrete column.

The central channel in columns of the new type may serve many purposes, such as those of ventilation or of the passage of pipes, or, with suitable lagging, for heating purposes, if provision be made for suitable modifications where junction is made with the floors, etc.

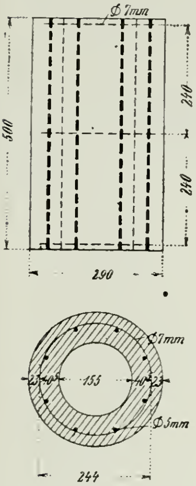


Fig. 6. Concrete Cylinder Reinforced with Rods and Loops.

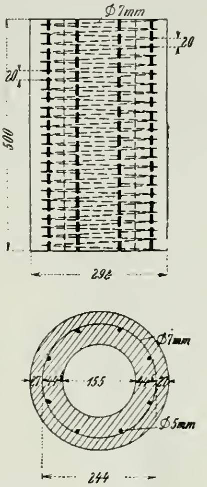


Fig. 7. Column with Spiral Reinforcement.

The experiments described justify conclusions which are best expressed by the diagrams *Figs.* 11 and 12. *Fig.* 11 shows the effect of a vertical pressure on an ordinary reinforced concrete column of diameter *d* and on a spirally reinforced column with turns $d/7$ apart. In the first case the shaded portions are broken away, whilst in the second this is prevented by the spirals, thus increasing the strength. *Fig.* 12 shows two columns constructed on the new system. In the first the spirals are just close enough to utilise the crushing strength of the concrete; in the second the spirals are so close that an increased strength is obtained and a complete alteration of the statical properties is produced. The experiments have shown that the strength of a cast-iron column may be trebled in this way, and that the resulting column is as trustworthy, as fire-resisting, and as elastic as if of steel.

The question whether cast-iron or steel tubes should be used depends both on technical and commercial considerations. The experiments show that both materials are quite trustworthy, and that the low elasticity of the cast-iron is no hindrance to its use. An important difference is that, whilst the load on steel is limited to its yield-point, in cast-iron it is the considerably higher crushing strength that matters.

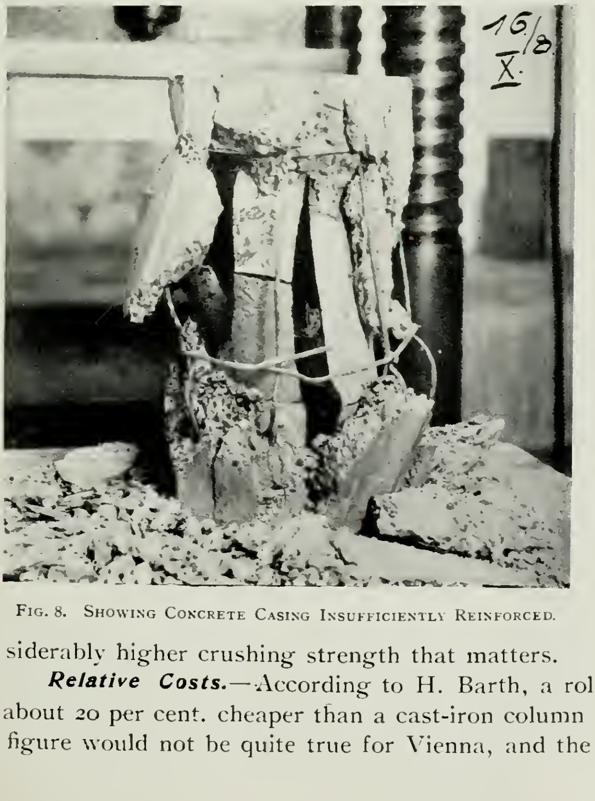


Fig. 8. Showing Concrete Casing Insufficiently Reinforced.

Relative Costs.—According to H. Barth, a rolled steel column in Berlin is about 20 per cent. cheaper than a cast-iron column of the same strength. This figure would not be quite true for Vienna, and the relative cost of transport of the two materials would vary in different cases. The following comparative estimates have been made by Messrs. Wagner, Biro, and Kurz, assuming columns 4 metres (13 ft.) high, with a central load of 68 tons and a pressure on the foundation of 30 kg./cm.² (427 lb. per sq. in.) :—

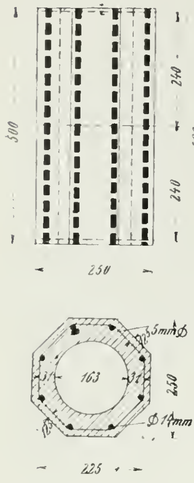


FIG. 9. Insufficiently Reinforced Column.

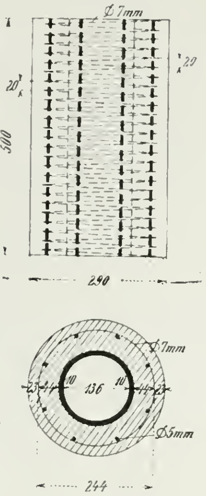


FIG. 10. Properly Reinforced Column.

	Price in Austrian Kronen.
1. Cast-iron column with base and capital. Factor of safety, 10; external dia., 240 mm.; thickness, 30 mm.; total weight, 910 kg. Price without fireproof covering	202·00
2. Mild steel column. Two No. 24 channels,][; weight, 540 kg., with factor of safety, 2·5. Price without fireproof covering	203·00
3. Reinforced concrete column, 40 cm. square, with 4 vertical rods and hoops at intervals of 40 cm. Price, including shuttering	92·40
4. Spirally reinforced octagonal concrete column, 40 cc. dia.	88·80
5. Cast-iron and concrete column on Dr. von Emperger's system, including the two sheet-iron cylinders for use as shuttering	66·58

The difference in cost is to be attributed mainly to the different stresses placed on the iron. Whilst in reinforced concrete the crushing strength of the concrete determines the stress in the steel, and in mild steel columns the yield-point is the determining factor, in the new system the full crushing strength of the iron is utilised, whilst preserving the same conditions of elasticity as with steel.

Computation of Columns.— The customary method of computing reinforced concrete columns, based on the average ratio of elasticities of the two components ($n = 15$), does not determine the factor of safety, as in the computation of beams. The author has previously shown, in his researches extending from 1902 to 1908, that a better method may be based on the breaking load. He assumes 5 as an entirely sufficient factor of safety for such columns, and in this is confirmed by Professor Rudeloff. Column 7, which broke under 184 tons, should thus have a permissible load of 36 tons; whilst the Austrian official method of computation only allows 21 tons, or a factor of safety of nearly 9, with the inconsistency that the concrete is only stressed to one-eleventh of its limit and the steel to one-sixth. The method is, therefore, to be rejected when, as in this case, the percentage of metal is large, and especially when a metal of low elasticity, such as cast-iron, is used, when n cannot be taken as more than 10.

An accurate computation shows that the cast-iron should be treated as if its crushing strength were the yield-point of an elastic material. The Austrian authorities have so far recognised the inapplicability of the old method to the computation of these new columns as to direct that they are not to be treated as reinforced concrete at all, and, although objections have been raised to the author's mode of computation, no alternative has been proposed.

In order to determine the dimensions of a column to carry a given load, P, it is first necessary to decide whether allowance is to be made for bending. The Prussian regulations state that this is unnecessary if $l = 18d$, that is, $l \leqq 60i$—where i is the radius of inertia, and this is confirmed by numerous experiments. The conclusion is that with a column 3 metres (9 ft. 10 in.) long the diameter

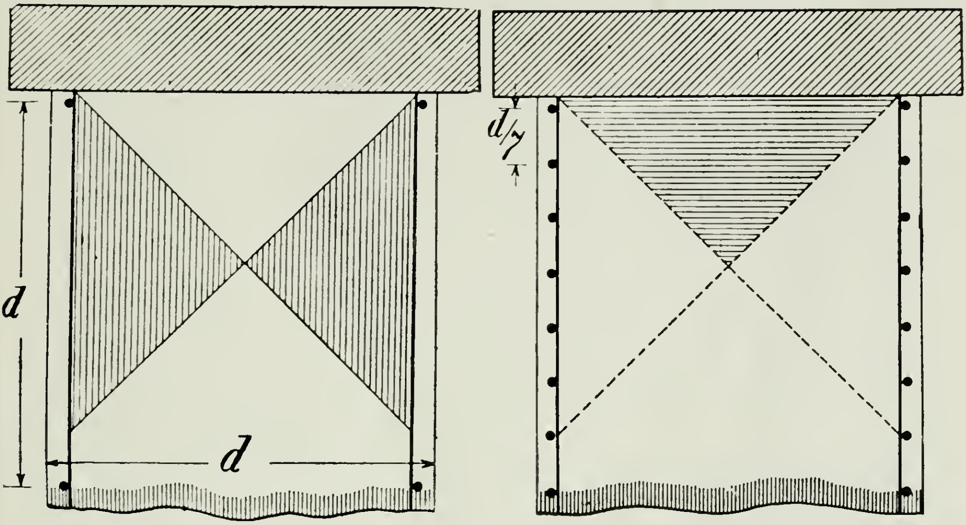


FIG. 11. CONCRETE REINFORCED WITH HOOPS AND SPIRALS.

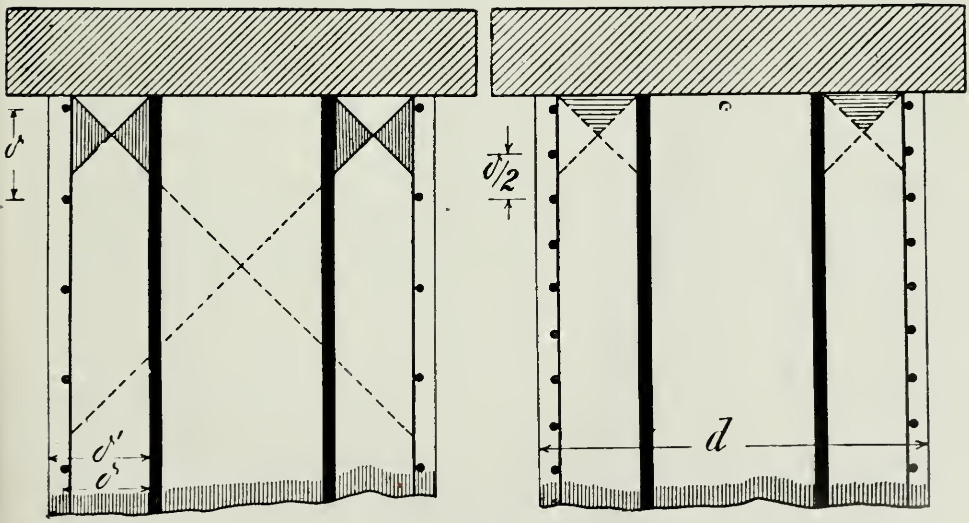


FIG. 12. DR. v. EMPERGER'S SYSTEM.

may be anything above 17 cm. (6·7 in.), without any correction for bending becoming necessary. The load is actually carried by two distinct columns, one of cast-iron and the other a spirally reinforced hollow concrete column, and these may be considered as actually separated—say by a thin layer of air or a coat of red lead. The carrying power should then be the sum of those of the two columns separately. This is shown by the following experiments: A cast-iron column broke under 209 tons, and a hollow reinforced concrete cylinder under 94 tons. The sum of these is 303 tons; and a column which was very nearly a combination of the two broke under 267 tons, the deficiency of 10 per cent. being accounted for by a slight difference in the measurements. In the two other cases the expected values were $94+274=368$ tons and $106+70=176$ tons, whilst the values actually obtained were 332 and 184 tons respectively, the differences lying within the limits of error.

We must, therefore, assume for the new system a uniform factor of safety, and we may conveniently fix this at 5, as for reinforced concrete. It is true that the Prussian regulations demand a factor of 10 as respects the crushing strength, but many experiments have shown that this is never actually attained.

We thus fix the permissible stress in the central column, if of steel, at $\frac{2400}{5}=480$ kg./cm.² (6,832 lb. per sq. in.), or for cast-iron at 1,000 to 2,000 kg./cm.², or, to allow an ample margin, at 750 to 1,500 kg./cm.² (10,750 to 14,336 lb. per sq. in.). Starting with these data, the calculation is carried out as follows:—

The column is 3 metres (9 ft. 10 in.) high, with a central load of 53 tons. Bending may be neglected. The cast-iron column is 14·4 cm. (5·7 in.) in external diameter, and 40·1 cm.² area (6·2 sq. in.). Being ordinary commercial cast-iron, we only take its crushing strength as 5 tons per sq. cm., so that the permissible load is $40·1 \times 750=30$ tons. There remain 23 tons to be carried by the hollow concrete column. If the concrete may be loaded to 28 kg./cm.², this may be doubled by reinforcement, giving 56 kg./cm.². Then:

$$23000=56\left(\frac{D^2-14·4^2}{4}\right)\pi\,;\quad D=\sqrt{1·27\frac{P}{\sigma_b}+d^2}$$

This gives D = 27 cm., or a thickness of concrete of 6·3 cm. (2·5 in.) between the cast-iron and the external reinforcement.

The diameter of the external spirals is thus 27 cm., and that of the column 29 cm. (11·4 in.). The newer Austrian regulations allow the external shell to be considered as carrying load up to 80 per cent. of its area. We therefore increase the 27 cm. to 29 cm., to afford protection to the spirals against fire, and have:

$$P=\sigma_b\,(F_b+15F_e+30F_s)=28(498+15\times 1·6+30\times 16·7)=$$
$$28\,(498+24+500)\text{ or}$$
$$P=28·6\text{ tons}$$

But as the iron column can carry 30 tons, and the concrete $498\times 2\times 28=$ 27,900 kg., the sum $30+28=58$ tons.

The column actually constructed with these dimensions broke under a load of 342 tons, so that a factor of safety of about 6 is obtained.

The following method is more general:—The total load P is divided into two approximately equal parts, and for one of these a suitable iron section with 750 kg./cm.² is selected. A reinforced concrete hollow column is then computed to carry the remainder, and the reinforcement is spaced according to the thickness found for the walls, remembering that the spacing of the spirals must not exceed that thickness, and 1 or 2 cm. is then added to the diameter.

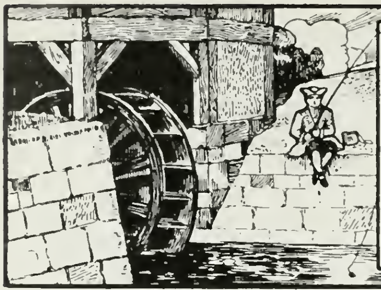
Construction of Arched Bridges.—The method of construction just described is suitable for many kinds of members, and especially for arches carrying a roadway, as the construction combines the advantages of convenience in erection and low dead weight of steel with the absence of riveting, the possibility of a simple, massive design, and the low cost of upkeep of reinforced concrete. The cost of large bridges is thus reduced to a minimum.

A scheme is described for a new bridge over the Danube Canal, with a span of 60 metres. The bridge is skew, its axis being inclined to that of the canal by 71° 48′ 29″. The height is only 4·2 metres, or 1-15, and this, combined with the loading conditions, introduces special difficulties of design.

In order to provide against eccentric loading, the haunches must be suitably strengthened at certain points. This is conveniently effected by using two separate iron tubes, and bringing these nearer to one another or separating them as required. In the design given the ribs are only so far thickened as to ensure that only compressive stresses, and not tensile, arise in the iron. The cast-iron tube is therefore replaced by two ⌓-shaped tubes, the distance apart of which is suitably chosen at the quarter-span, whilst they unite in a single cast joint at the hinges. The calculation is made as for a column, except that n, instead of being the ratio of the elasticities, is now the ratio of the breaking loads of the two materials under compression—for instance, $n = \frac{5000}{200} = 25$. That is, the assumption is made that when deformation occurs before fracture, a stress of 5,000 kg./cm.^{2} in the iron and of 200 kg./cm.^{2} in the concrete is reached simultaneously. The value 25 is merely provisional.

The Austrian official regulations fix as the permissible stress in a concrete mixture of 350 kg. of Portland cement with 1 cubic metre of mixed sand and gravel the value 29 kg./cm.$^2 + 0{\cdot}2s = 41$ kg./cm.^{2}, s being the span in metres, in this case 60. The stress in steel reinforcing rods would then be $15 \times 41 = 615$ kg./cm^2, this being assumed to give a factor of safety of 5. For a good quality of cast-iron we may then allow, assuming the same factor, $7{,}000/5 = 1{,}400$ kg./cm.^{2}. As a concession to official authorities of a highly conservative tendency a lower figure may be taken, say 1,000 kg./cm^2, or the bridge may be computed as if entirely of iron, neglecting the additional strength given by the concrete.

The reinforced concrete method of construction is now very frequently resorted to in competitions for bridges. The proposed method has the advantage over steel of obviating the necessity for costly riveting. The iron sections are screwed, and form a self-carrying structure, to which the concrete coating is affixed without difficulty. The method may also find application in the construction of arches, domes, electric light masts, towers, etc. The author believes it possible to construct arched bridges of greater span than heretofore, and halls, such as those of large shops, in a slenderer form than has yet been attained. Further experiments are planned, which the author will communicate to all interested persons.



TWO LARGE REINFORCED CONCRETE RESERVOIRS.

In the article below we are describing two important reinforced concrete reservoirs in this country which have recently been erected, and which show the suitability of this method of construction for work where stability and economy are required.—ED.

RESERVOIR FOR THE OLDHAM CORPORATION.

THIS reservoir forms part of a scheme which was designed, under powers obtained by the "Oldham Corporation Act, 1909," to utilise the waters arising from the disused workings of the Butterworth Hall Colliery. By an agreement between the Company and the Oldham Corporation, the waters will be delivered

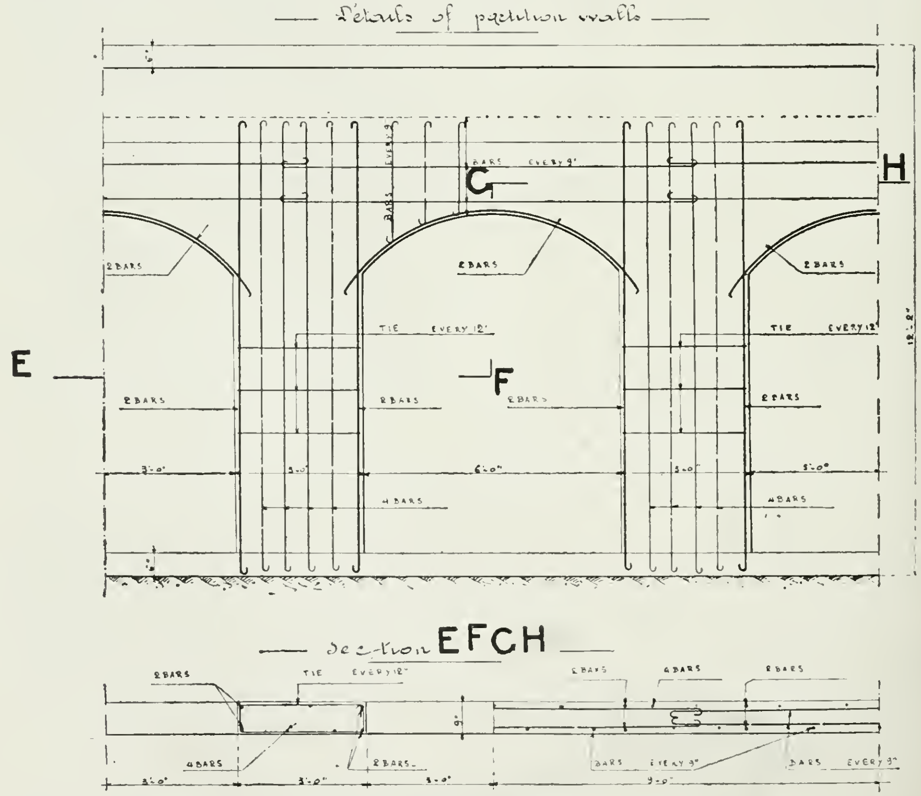


Fig. 1. Details of Partition Walls.
REINFORCED CONCRETE RESERVOIR FOR THE OLDHAM CORPORATION.

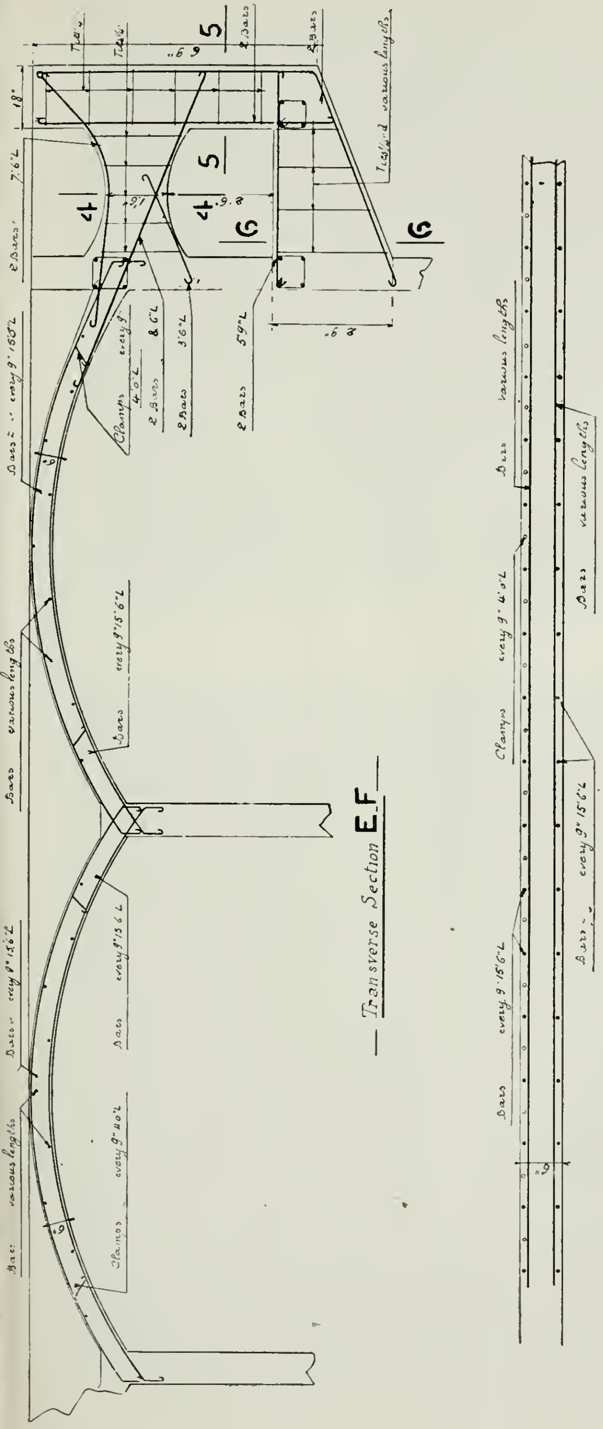


Fig. 2. Detail showing Flying Buttresses.

Reinforced Concrete Reservoir for the Oldham Corporation.

into the storage reservoir of the Corporation, and thence into Piethorne Reservoir, from which they will be conveyed into service through the existing mains.

The reservoir, which is entirely underground, has a capacity of 225,261 gallons. It has an irregular shaped plan, with a maximum length of about 112 ft. and a maximum width of 60 ft. It is divided into nine separate compartments, having a depth of 12 ft. 2 in., including the thicknesses of the top and bottom, the divisions occurring about 12 ft. apart. The external walls are 9 in. and 12 in. thick, and designed in such a manner that they are strong enough to safely resist the water pressure when the reservoir is full, without any assistance from the earth pressure on the outside; and this, of course, is the only satisfactory way of constructing the work.

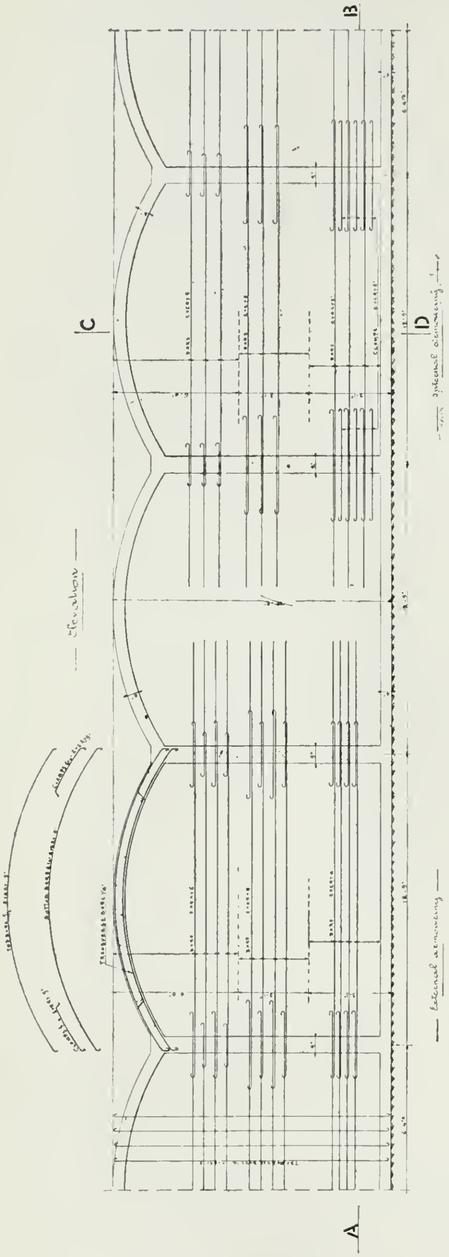


Fig. 3. Reinforcement for External Walls.
Reinforced Concrete Reservoir for the Oldham Corporation.

The reinforcement in these walls consists of plain round rods horizontally and vertically at varying distances apart, as indicated on the drawings.

The internal divisions are 9 in. thick and formed with arched openings about 6 ft. wide and 8 ft. high. These arches are reinforced with two 7-16-in. diameter bars, while the wall between the openings is reinforced with four 5-16-in. diameter bars placed vertically, and one 7-16-in. diameter bar at each angle of the pier, with horizontal 5-16-in. diameter ties at 12-in. centres.

The compartments themselves are arched with 6 in. of concrete, reinforced with radial bars every 9 in. in the top and bottom surfaces, and transverse bars.

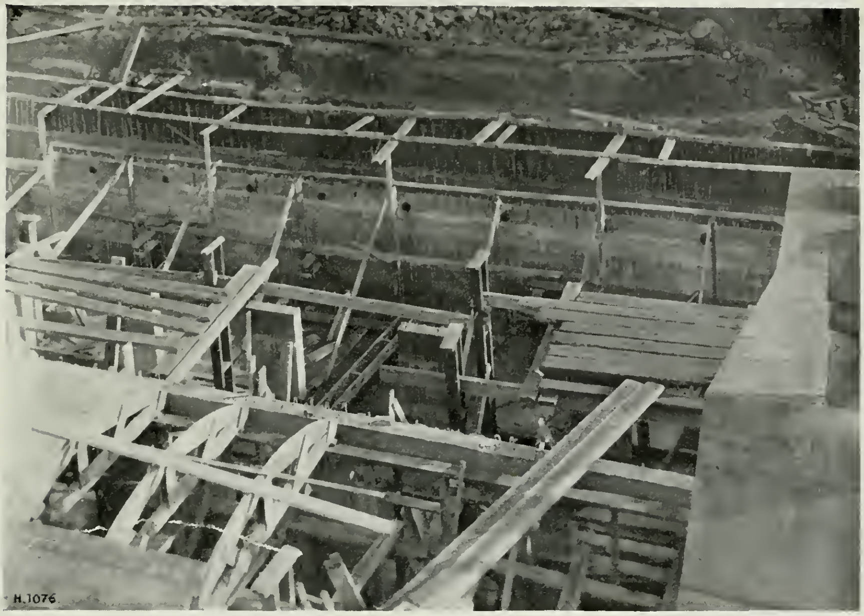
It is interesting to note the method of resisting the thrust from the end of the series of arches, by a series of flying buttresses, as illustrated by the drawing in *Fig.* 2, this extending for the full width of 60 ft. and rendering the end wall safe against overturning in the event of subsequent excavation at this point.

The bottom slab was formed with 6 in. of concrete, reinforced generally with bars at 9-in. centres. The whole of the parts are well tied together and clamps provided at the various junctions to ensure a monolithic construction and well distribute the stresses.

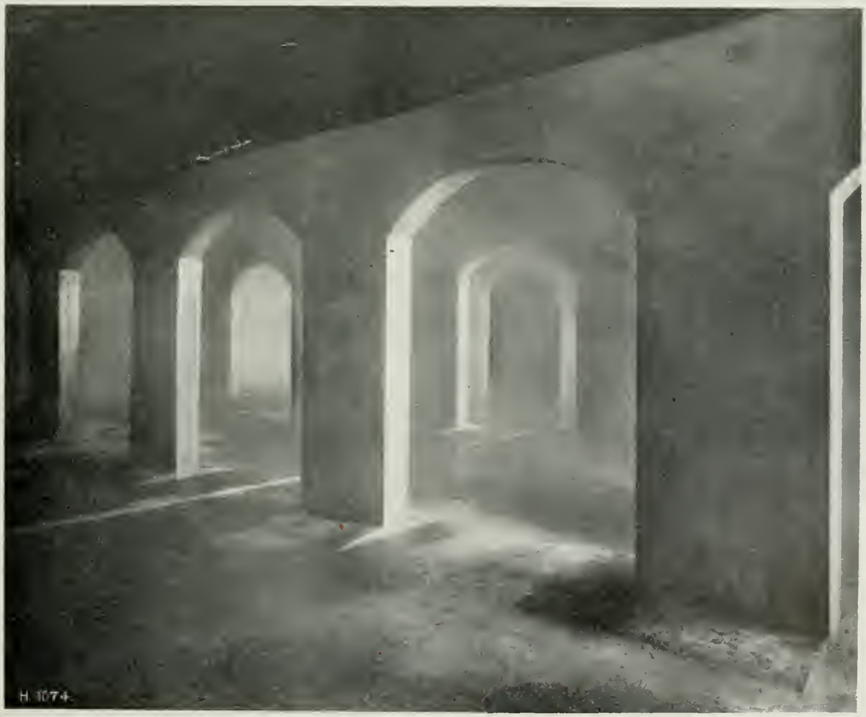
The weight of steel used for the reinforcement has been 10 tons, the number of rods 6,700, and their total length 66,900 ft.

The reinforced concrete details of the reservoir were designed by the Considère Construction Co., of Victoria Street, S.W., and, owing to the irregular shape, it presented more difficulty than the usual type of reservoir.

The engineer for the work was Mr. C. J. Batley, M.Inst.C.E.



Reservoir in course of construction.

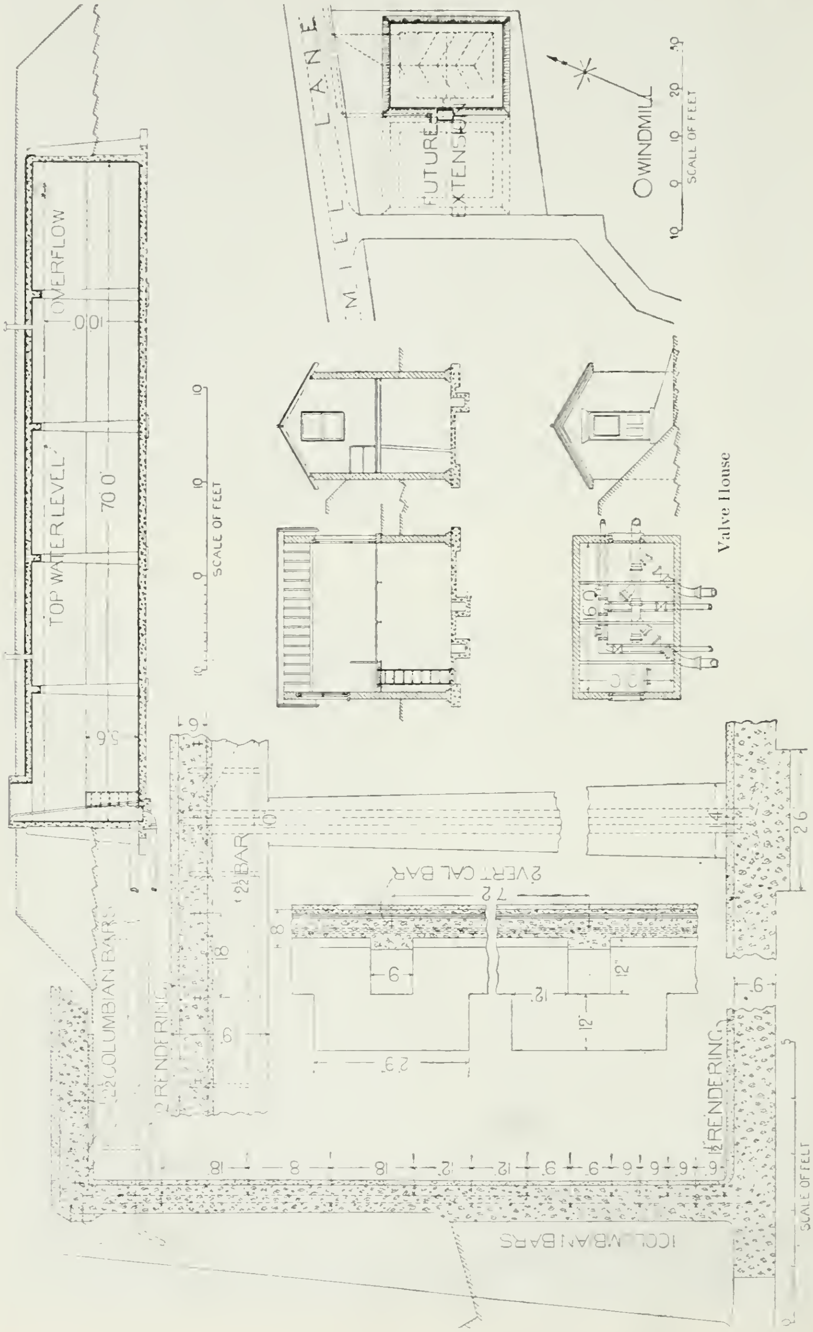


Interior of Storage Reservoir.

Reinforced Concrete Reservoir for the Oldham Corporation.

REINFORCED CONCRETE RESERVOIR AT SKEGNESS WATERWORKS.

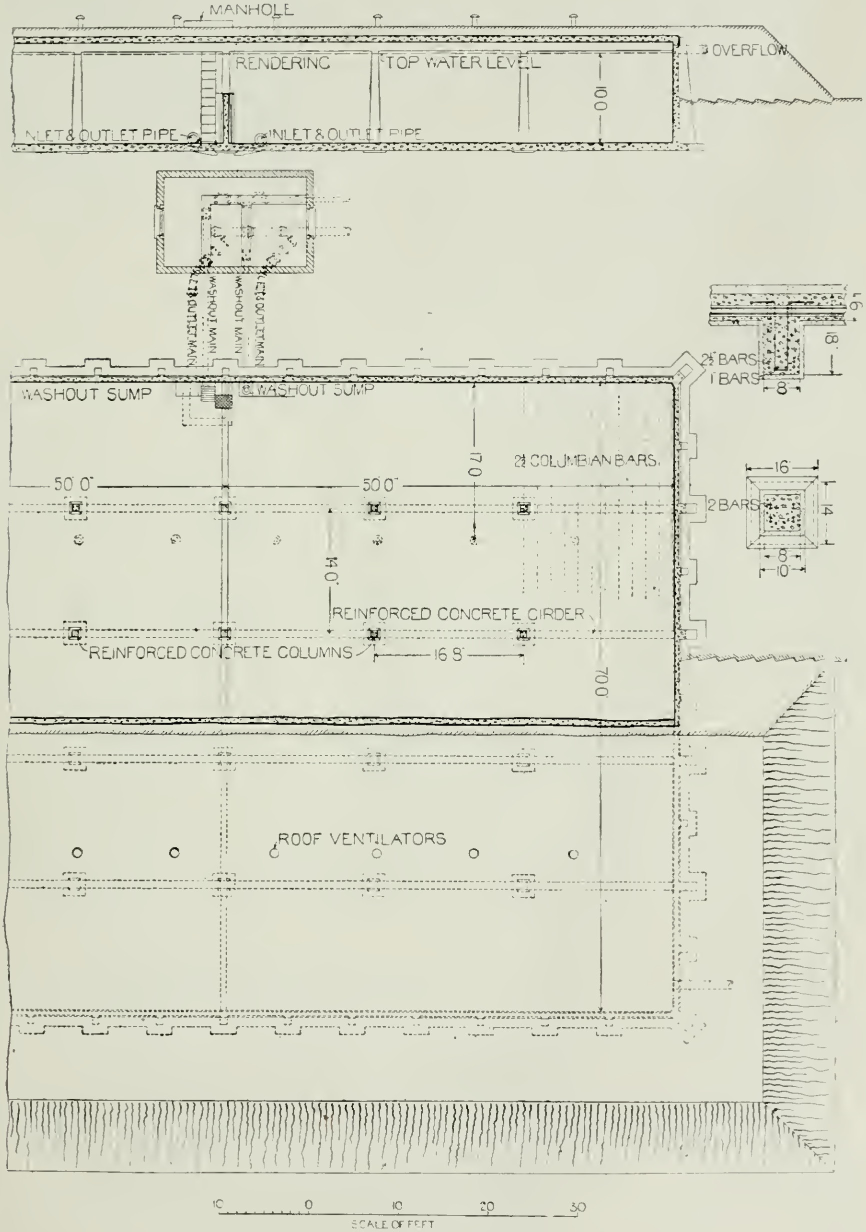
The reservoir described below was constructed in connection with the new Skegness Water Works, which were designed and carried out by Mr. Percy



Section and Details.

Reinforced Concrete Reservoir at Skegness Waterworks.

Griffith, M.Inst.C.E., and which were recently described by him in a paper read before the Institution of Water Engineers, from which the following particulars are taken :—



Plan, Section and Details.

Reinforced Concrete Reservoir at Skegness Waterworks.

A point of special interest in connection with these waterworks was the adoption of reinforced concrete for the service reservoir. This was constructed on a hill top a mile and a quarter west of the pumping station, the elevation being 200 ft. above O.D., ample to give the necessary pressure at Skegness, allowing for friction in the main. The capacity is 400,000 gallons when full, and the reservoir is divided by a transverse partition 5 ft. 6 in. high, into two parts, to provide for cleaning out the floor without throwing the reservoir out of use. To avoid the expense of laying a double line of main between the pumping station and the reservoir, the rising main enters at the bottom, and serves as an outlet main as well; in fact, during the time that pumping is in operation the supply does not enter the reservoir except so far as the rate of pumping exceeds the demand at Skegness. No inconvenience has been experienced from the resulting risk that a leak in the main might possibly empty the reservoir, as, by providing proper supervision, assisted by automatic alarms at suitable points, the engineer has promptly dealt with the leaks that have occurred long before any loss of importance has taken place.

The system of reinforcement adopted is the + and H bar, securely wired at inter-sections, the roof being supported by concrete girders reinforced along lines of tensional stress. The internal dimensions are 100 ft. by 70 ft. in plan, with 10 ft. depth of water. External walls are 8 in. thick at base and 6 in. thick at top, supported at 7 ft. centres by external counterforts 12 in. wide, tapering from 1 ft. 9 in. at base to 9 in. at top (additional to thickness of walls). The floor was not reinforced, and is 9 in. thick, extending 12 in. beyond external walls, and thickened to 12 in. under the piers. These are 16 in. square at base, tapering to 10 in. square at top, and are arranged at 16 ft. 8 in. centres along the major axis, and 14 ft. centres on the minor axis. The roof is 6 in. thick, supported on concrete girders 12 in. deep (additional to thickness of roof) by 8 in. wide.

The top of the roof is rendered with 2 in. of cement, the inside of the walls with 1 in., and the inside of the floor with 1½ in. of cement, the whole of this work being a model of what rendering should be. There has been no leak whatever since completion, although the floor developed a crack during construction, owing to the existence of a fissure (unobserved during the excavation) in the underlying chalk. This was readily repaired by concrete rammed into the fissure, the floor concrete being strengthened by old rails (laid transversely to the crack) bedded in the concrete.

As regards the use of reinforced concrete for service reservoirs, the author stated that he was satisfied that, as regards economy and stability, the system is far more satisfactory than ordinary brickwork or concrete construction. The cost in this case (£2,309) was, perhaps, somewhat below market prices, as the contractors were very anxious to secure the order; but, even so, the author is satisfied that ordinary construction of equal stability would have cost more than did the design adopted. As regards water-tightness, the author does not consider that reinforcement gives any material advantage, the result depending entirely upon the efficiency of the rendering; but in this instance there is no doubt that the work was carried out so efficiently that the reservoir may be looked upon as a model for imitation.

DESIGN OF T-BEAMS IN REINFORCED CONCRETE.

By R. N. MIRZA, B.Eng., M.Inst. Min.E., Assoc. G.W.T.C.

The following article on the design of T-beams will be found interesting by those connected with the design of reinforced concrete structures.—ED.

The charts accompanying this article give a rapid method of designing T-beams and calculating their resistance to stresses induced by bending. By the aid of these diagrams, the required values of depth, breadth, reinforcement, etc., can be determined accurately, when the bending moment on the structure is known. No approximate formulæ are used to calculate the diagrams.

In order to justify the correctness and utility of the diagrams it is necessary to work out fully the deductions, from the fundamental equations, according to which the curves have been drawn.

Although the mathematical working out of the deduced equations is somewhat complicated, application of the final results and the reading of the charts for designing and checking purposes will be found to be extremely simple and should prove invaluable for rapid and accurate calculating purposes.

The following nomenclature has been used :—

M=Bending moment in inch-lbs.
t=Tensile stress in metal in lbs. per square inch.
c=Compressive stress in concrete in lbs. per square inch.
z=Distance of resultant thrust in concrete from compressed edge of beam in inches.
n=Distance of neutral axis from compressed edge in inches.
K=Ratio of the depth of neutral axis to the effective depth of the beam.
d=Effective depth of beam in inches.
d_s=Depth of slab in inches.
A_c=Area of concrete in compression in square inches.
A_t=Area of metal in tension in square inches.
$Y=\left(\frac{d}{d_s}\right)=\frac{\text{Effective depth of beam}}{\text{Depth of slab}}$
$m=\frac{E_s}{E_c}$ the ratio of coefficients of elasticity of steel and concrete=15.
C, G, ϕ, g, are constants used in the equations.

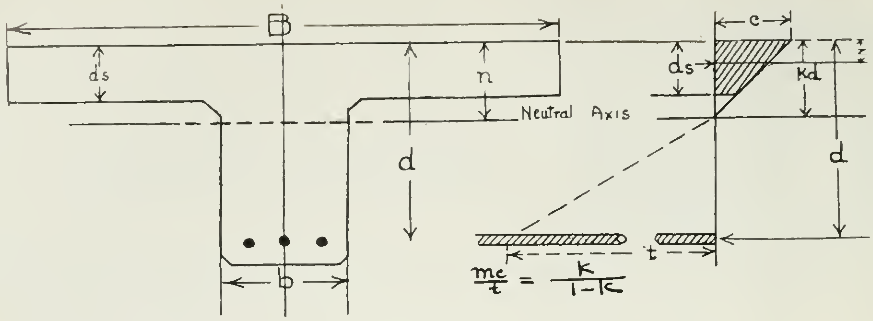
Two distinct cases arise in the design of T-beams :—

(*a*) When the thickness of the slab is greater than n.
(*b*) When the thickness of the slab is less than n.

In both cases

$$\frac{mc}{t}=\frac{K}{1-K}$$

Case (*a*).



Equating the moments of resistance to the bending moment,

$$M = A_t td\ (1-\tfrac{1}{3}K) = \frac{c}{2}Bd^2K\ (1-\tfrac{1}{3}K)$$

and $$z = \tfrac{1}{3}Kd = \frac{n}{3}$$

Substitute for d, Yd_s $\left(Y = \dfrac{d}{d_s}\right)$

then $$z = \tfrac{1}{3}KYd_s$$
$$M = tA_tYd_s\ (1-\tfrac{1}{3}K)$$
$$A_t = \frac{M}{\left\{td_s\ (1-\tfrac{1}{3}K)\right\}Y} \quad . \quad . \quad . \quad . \quad . \quad A$$

Similarly $$M = \frac{c}{2}Bd_s^2Y^2K\ (1-\tfrac{1}{3}K) \quad . \quad . \quad . \quad . \quad . \quad . \quad B$$

In designing t and c are assumed and hence K is known therefore

$$A_t = \frac{M}{\text{constant} \times Y}$$
$$M = \text{Constant} \times Bd_s^2Y^2$$

Average stress on concrete $= \dfrac{c}{2}$

CASE (b).

Neglecting the small compression in the web between d_s and n, the mean compressive stress on the flange is

$$\tfrac{1}{2}\left\{c + c\frac{Kd-d_s}{Kd}\right\} = \frac{c}{2}\left\{\frac{2Kd-d_s}{Kd}\right\}$$
$$= \frac{c}{2}\left\{\frac{2KY-1}{YK}\right\} = \left\{2-\frac{1}{KY}\right\}$$

It is evident that whenever Y is greater than $1/K$ the neutral axis lies above the soffit of the slab.

$$z = \frac{d}{3}\cdot\frac{3Kd-2d_s}{2Kd-d_s} = \tfrac{1}{3}\left[\frac{3YK-2}{2YK-1}\right]d_s$$

Equating the moments,

$$M = A_tt\ (d-z) = \tfrac{1}{2}\ Bdc\frac{2Kd-d_s}{Kd}\ (d-z)$$

$$A_tt\ (d-z) \qquad = 2d_s\left\{\frac{KY^2 - \cdot 5Y\ (1+K) - {}^2/_3}{2KY--1}\right\}tA_t$$

denoting $$2\left\{\frac{KY^2 - \cdot 5Y\ (1+K) - {}^2/_3}{2K\theta - 1}\right\} \text{ by } g$$

Resisting Moment $= M = A_td_sgt$

and calling $gt = G$

$$M = GA_td_s$$

also
$$M=\frac{c}{2}Bd\,\frac{2Kd-d_s}{Kd}\,(d-z)$$

substituting Yd_s for d,

$$M=c\left\{\frac{Y^2-\cdot5Y\left(\frac{1+K}{K}\right)-\frac{1}{3K}}{Y}\right\}Bd_s^2$$

and putting $\phi=\dfrac{Y^2-\cdot5Y\left(\frac{1+K}{K}\right)-\frac{1}{3K}}{Y}$

$$M=c\Phi Bd^2$$

which may be further simplified in the form

$$M=CBd_s^2$$

Hence the following important equations are deduced :—

(*a*) When the neutral axis is within the slab—*i.e.*, from $Y=1$ up to $Y=1/K$

$$M=\frac{c}{2}Bd_s^2Y^2K(1-\tfrac{1}{3}K)\text{ and}$$

$$A_t=\frac{M}{td_s(1-\frac{1}{3}K)Y}$$

(*b*) When the neutral axis is within the slab—*i.e.*, from Y greater than $1/K$ and upwards— $M=CBd_s^2$ and

$$A_t=\frac{M}{G.\,d_s}$$

The accompanying curves are drawn giving values of C, z, G, g, ϕ, and average value of c with respect to Y from Y equal 1 upwards.

It will be observed that the first portion of each of the curves follows the equations for Case (*a*) giving values up to $Y=1/K$, and then according to the equations deduced for the case when Y is greater than $1/K$, and it will be noticed that there is a slight "cusp" at the stage at which the neutral axis of the beam passes outside the slab and is denoted on the curves by heavy dots.

METHOD OF DESIGN.

In designing a T-beam the following quantities are generally known— the depth of the slab, the bending moment, and, B, the breadth of slab acting with the beam. Having decided upon the stresses to be permitted on steel and concrete, K is at once obtained either from the fundamental equation $\frac{mc}{t}=\frac{K}{1-K}$ or by referring to the chart given on page 534, R.I.B.A. first report on Reinforced Concrete.

The resisting moment of the T-beam is given by the formula $RM=CBd_s^2$

Equate the resisting moment to the bending moment

$$M=CBd_s^2$$

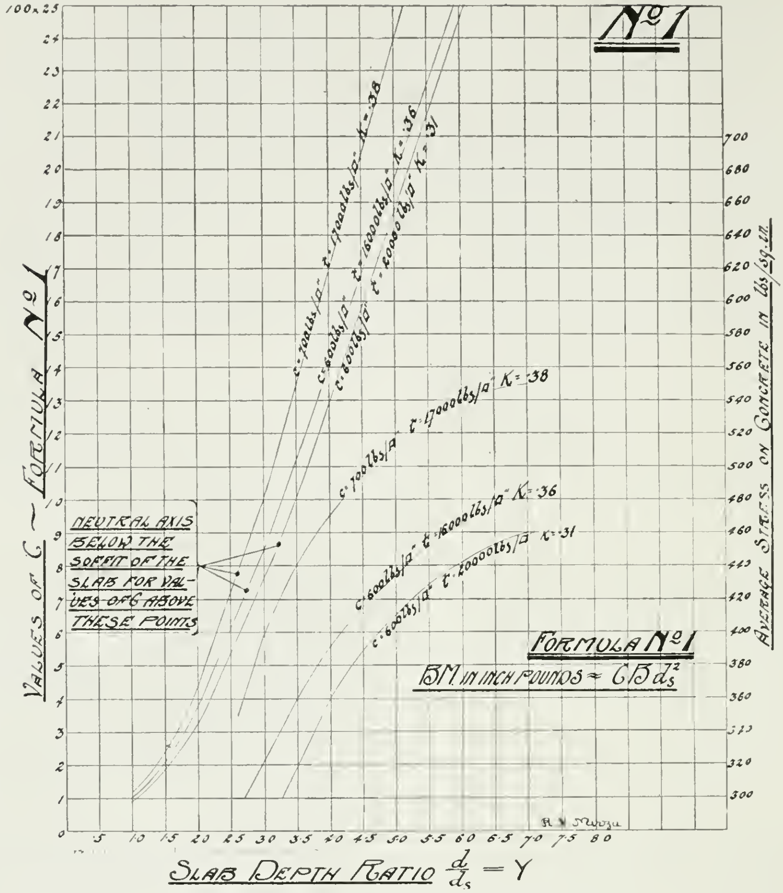
Calculate C from this equation, and from curves No. 1 connecting C and Y (curves are given for the three most usual values of K) the value of Y is directly obtained. If the value of C lies above that marked by the heavy dot, the neutral axis lies below the soffit of the slab.

Having obtained Y, it is evident that the effective depth of the T-beam from the top of the slab to the centre of steel in tension is equal to $d=Yd_s$

Trace the corresponding value of G for the value of Y on curves No. 2.

Area of metal required in tension $=A_t=\dfrac{M}{G.d_s}$

Curves are also given for rapidly obtaining values of z and average stress on concrete.



METHOD OF CHECKING DESIGNS.

The checking curves apply to designs which have been prepared on the assumption that stresses in steel and concrete do not exceed 16,000 lb. per sq. in. and 600 lb. per sq. in. respectively, but similar curves can be easily prepared for checking designs in which other stresses have been adopted.

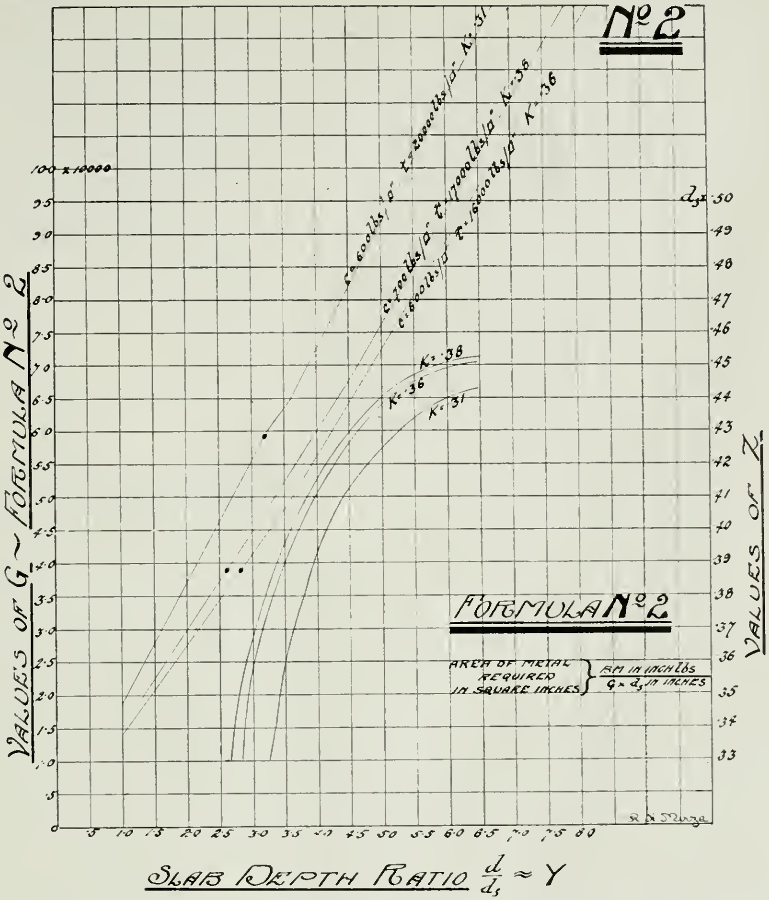
In a design which has to be checked the following quantities are known—

A_t, Bending Moment. B, d, d_s, and hence $\frac{d}{d_s}$ i.e. Y

From the ϕ curve the value of ϕ for the given value of Y is obtained; then c, the compressive stress on concrete, $= \frac{\text{Bending Moment}}{\phi\, Bd_s^2}$

Similarly, to find t, the tensile stress on steel, obtain the corresponding value of g for the value of Y, then $t = \frac{\text{Bending Moment}}{g \cdot A_t \cdot d_s}$.

Example.—Required, to find the depth of a T-beam and the area of metal to resist a bending moment of 1,500,000 in.-lb. Depth of slab 5 in.; the avail-



able breadth of slab acting with beam 60 in. Stresses on concrete and steel to be fully developed. Assume c = 600 lb. / □" & t = 16,000 lb. / □").

$$M = CBd_s^2 \qquad C = \frac{1.500,000}{60 \times 25} = 1,000$$

Y is found from curve No. 1 to be equal to 3·3

∴ the effective depth of the beam = $3{\cdot}3d_s$

$$= 3{\cdot}3 \times 5 = 16{\cdot}5''$$

From curves No. 2 the corresponding value of G for Y = 3·3 is 4·6 × 10,000

therefore

$$A_t = \frac{1,500,000 \text{ inch lbs}}{G \times d_s} = \frac{1,500,000}{4{\cdot}6 \times 10,000 \times 5}$$
$$= 6{\cdot}52 \text{ sq. in.}$$

In this method it is noticed that the designer does not require to assume the depth, but finds at once the depth which will develop the full stresses.

To justify these solutions, same results as above are worked out below from the fundamental equations upon which the preparation of the accompanying curves is based.

Stresses to be developed 600 lb. □" & 16,000 lb. □" K = ·36. $d_s = 5''$.

Assume a total effective depth of 16·5 in. (and the difficulty lies in assuming the correct depth that will develop full stresses and utilise the whole available breadth of the slab).

$$\text{Lever Arm} = d - \frac{d_s}{3}\left(\frac{3n - 2d_s}{2n - d_s}\right)$$ R.I.B.A. second report, page 24.

$$= 16{\cdot}5 - \frac{5}{3}\left(\frac{3 \times 5{\cdot}94 - 10}{2 \times 5{\cdot}94 - 5}\right)$$

$$= 16{\cdot}5 - 1{\cdot}9 = 14{\cdot}6$$

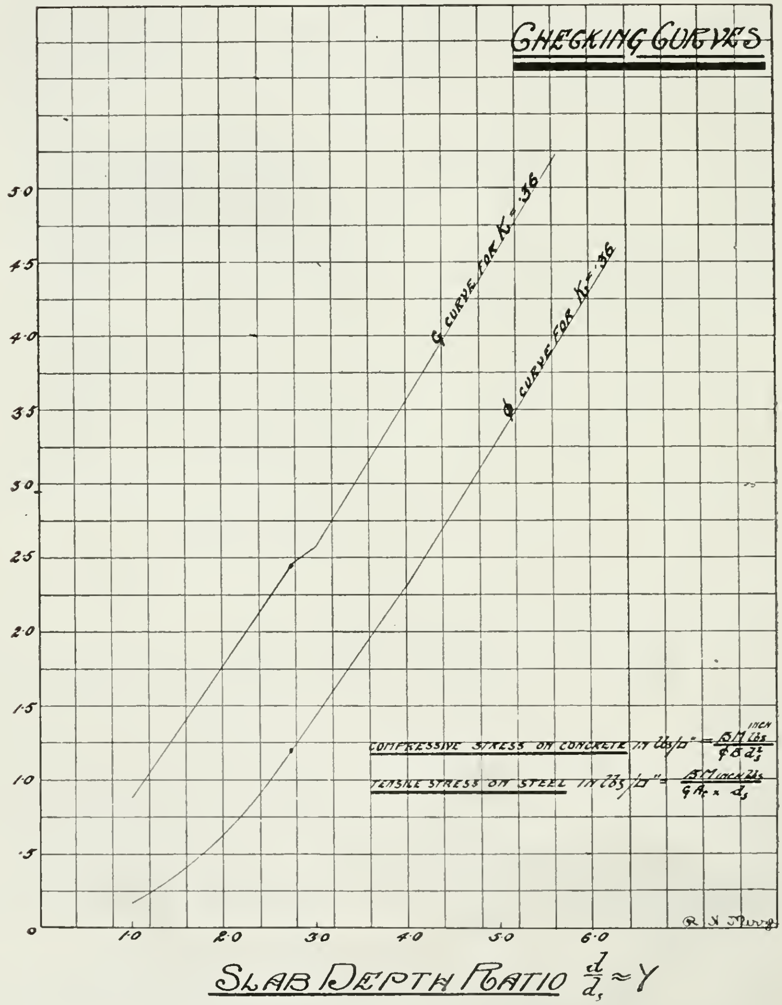
$$A_t = \frac{1{,}500{,}000}{16{,}000 \times 14{\cdot}6} = 6{\cdot}4 \text{ sq. in.}$$

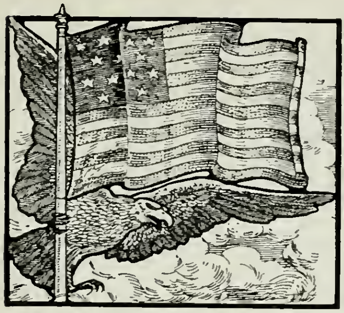
c = the compressive stress on concrete

$$= \frac{2\,Mn}{Bd_s\,(2n - d_s) \times \text{Lever Arm}}$$

$$= \frac{2 \times 1{,}500{,}000 \times 5{\cdot}946}{60 \times 5 \times (11{\cdot}88 - 5) \times 14{\cdot}6}$$

$$= \frac{59{,}460}{100{\cdot}2} = 593{\cdot}4 \text{ lb./}\square''$$





USE OF REINFORCED CONCRETE IN SAN FRANCISCO AND VICINITY.

By JOHN B. LEONARD, M.Am.Soc.C.E.

The following article on the use of reinforced concrete in San Francisco is a summary of a paper read some time ago by Mr. John B. Leonard, of San Francisco, before the National Association of Cement Users of the United States.—ED.

In order properly to appreciate the use of reinforced concrete in San Francisco and vicinity at the present time, it is essential to understand the conditions existing in this territory immediately previous to the great earthquake of San Francisco in 1906.

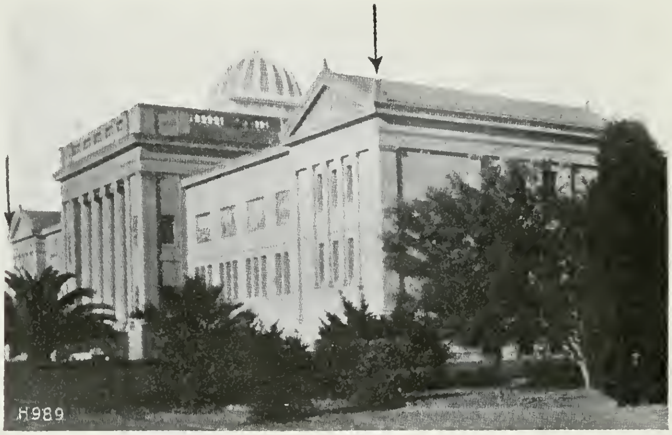
Municipal Laws of San Francisco.—Prior to April 18, 1906, the Building Laws of the City of San Francisco were so framed that they did not permit the construction of an all reinforced concrete building. They did allow the use of reinforced concrete for foundation work, sidewalk walls, and floor slabs. Architects were also allowed the use of reinforced concrete intermediate beams in a structural steel frame, and this device had been resorted to in two or three instances. There were many buildings in which reinforced concrete floor slabs were used, and also concrete fireproofing for the columns, but a large portion of the practice used tile for floor and fireproofing purposes.

The advocates of reinforced concrete, however, were making a very strenuous fight to secure permission to construct buildings of this material, and their efforts would undoubtedly have secured favourable results in a short time.

The Effect of Earthquake on Reinforced Concrete Buildings.—Owing to the Building Laws of San Francisco denying the privilege of constructing reinforced concrete buildings in the city limits there was no completed building of this type in San Francisco at the time of the earthquake and fire. There was one building just being constructed. The owner had never received an official permit for the erection of this building, but being confident of the merits of reinforced concrete, intended to abide by the issue in the courts. This structure was intended for storage purposes and was to be six stories high. The foundation, columns, girders, floor and roof slabs were to be of reinforced concrete, and the enclosure walls of brick. At the time of the great disaster the second floor had been completed and forms were up for the third floor, also the curtain walls were up nearly to the third floor.

The building was not subjected to any intense fire, but it was in a section of the city that suffered severely from the earthquake. An examination of the building a few days after the disaster showed that the brick curtain walls had suffered considerably, but that the reinforced concrete portion of the structure was entirely sound. The faults in the curtain walls have since been corrected and the building carried on to completion in reinforced concrete.

At the Leland Stanford, Jr., University, located at Palo Alto, about thirty miles south of San Francisco in the Santa Clara Valley, and but a few miles from the famous fault line, there existed a reinforced concrete building that was constructed some twenty-three years ago. This building, shown in Fig 1, known as the museum building, has a length of about 300 ft., and a depth of about 50 ft., and is two stories high. The construction was entirely of reinforced concrete, consisting of interior columns, girders, beams and floors, and enclosed with concrete bearing walls. The enclosure walls, however, were not reinforced, except at the floor levels, the space between the floor levels consisting of plain concrete.



The Leland Stanford, Jr., University, Museum Building, California.
REINFORCED CONCRETE IN SAN FRANCISCO AND VICINITY.

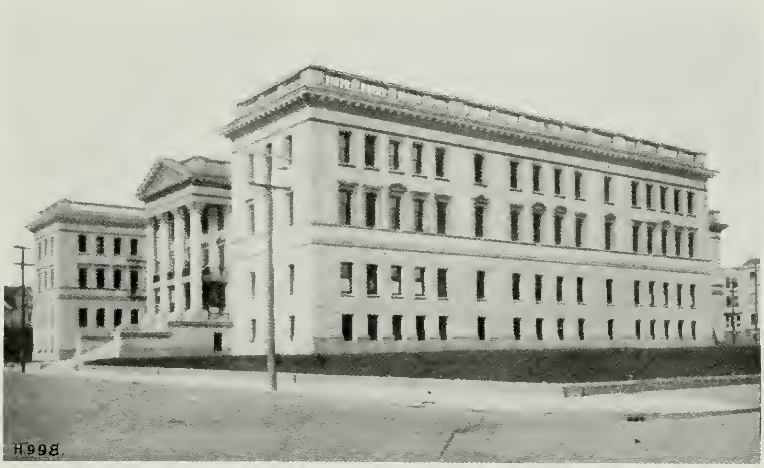
Shortly after the completion of the building, as was to be expected, shrinkage cracks appeared in some of the walls. Recent observers have mistaken these shrinkage cracks for earthquake cracks, but there is ample testimony of unquestionable character to prove that they were in existence years before the earthquake occurred.

It became the writer's privilege about a month after the shock to make a thorough minute examination of the structure in company with an engineer and an architect. Because of our realisation of the important part that reinforced concrete would probably take in the reconstruction of the stricken section, our examination was made with unusual care. At the completion of this task, it was the unanimous opinion that this structure had suffered no structural damage from the earthquake. It was not subjected to any fire test. Columns, girders and floors showed no cracks or defects of any character whatever. The only item of damage that could be found in the building at all was a hole that had been punctured in one of the floors, this puncture being caused by the falling of a piece of statuary.

That this building had been subjected to a severe vibratory motion was evident in the main entrance. This entrance had a marble wainscoting about 4 ft. high, and the floor was covered with chips and dust caused by the grinding of the edges of the marble used in the wainscoting. Some of the chips were as large as the thumb nail.

There was just being completed an addition to the museum building, this addition being almost ready for occupancy. The construction of the new work consisted of brick bearing walls and concrete floors, the concrete floors being supported by steel girders and columns. All the new work was badly wrecked by the earthquake. It is interesting to know that the University authorities had abandoned the reinforced concrete construction and resorted to this type as being safer. The alarm was caused by the shrinkage cracks that existed in the main museum building.

The writer has heard it stated, and is informed that the statement exists in print, that the Memorial Arch at the Leland Stanford, Jr., University was a reinforced concrete structure, and that it was a complete wreck. I take this opportunity to refute this statement, in so far as it pertains to reinforced concrete; the arch was a wreck, but it was constructed of brick.



Southern Pacific Railway Company's Hospital, San Francisco.
Reinforced Concrete in San Francisco and Vicinity.

Fire Resisting Qualities of Reinforced Concrete.—One of the great misfortunes for us is that there did not exist at the time of the fire an all reinforced building in San Francisco. Such information as was gleaned from the San Francisco fire on this subject was derived wholly from the behaviour of the concrete floors and fireproof columns in some of our steel frames. As in all other places, good, bad and indifferent concrete work had been done, so that the results must of necessity be variable in character. The same faults of workmanship existed, no doubt, in the buildings in which tile was the fireproofing element.

Re-building of San Francisco.—When San Francisco faced the problem of rebuilding it was a source of astonishment to those interested in construction to see the attention and investigation that was being given to reinforced concrete by the laymen. The public opinion became so strong for this material that the Board of Supervisors were compelled to recognise its merits and permit its use for all building operations.

Its superior fire-resisting qualities were recognised by comparing the general condition of such buildings as had used concrete with those which had used tile. The result is that to-day in San Francisco concrete has been used as fire-proofing for the steel-frame buildings almost exclusively.

The Sheldon Building, San Francisco.
REINFORCED CONCRETE IN SAN FRANCISCO AND VICINITY.

Up to June 30, 1910, there had been issued in San Francisco permits for 132 reinforced concrete buildings, and several permits have been issued since.

State of California.—The State of California, through the State Engineer's Department, has used reinforced concrete very extensively in the replacing of such State buildings as were destroyed by the earthquake. The State Insane Asylum, situated at Agnews, a place about twenty miles from the Leland Stanford, Jr., University, has been rebuilt of reinforced concrete.

Very extensive additions to the State Penitentiary at San Quentin, situated on the bay but a few miles from San Francisco, were made of the same material.

The State Board of Harbour Commissioners.—Since 1906 the State Board of Harbour Commissioners have used reinforced concrete very extensively in the construction of wharves. Piers 36, 38 and 40 are founded on concrete cylinder piers, which support a series of steel girders. These girders are encased in concrete, and on them rests a reinforced concrete slab which supports the entire traffic of the wharf. Pier 54, which is now under construction and nearly completed, is supported by concrete cylinders. The floor system of this pier, however, consists of a series of girders, beams and slabs, all of which are constructed entirely of reinforced concrete. This is the last pier constructed by the Board of Harbour Commissioners, but they now have in preparation plans for another one that will incorporate the same type of construction as pier 54.

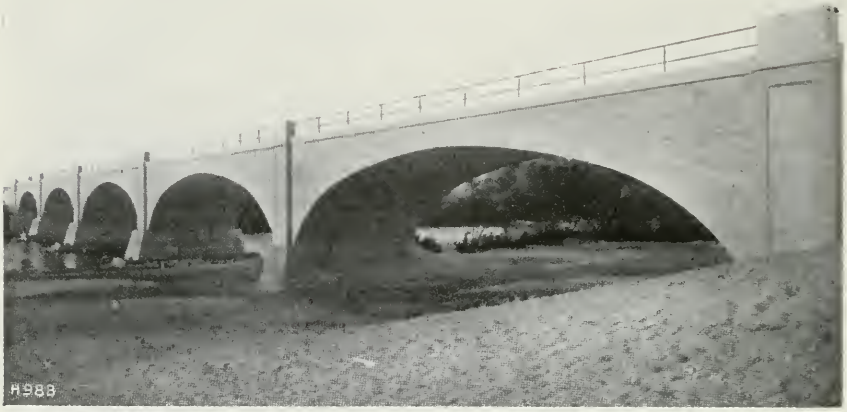
Reinforced Concrete Pier, San Francisco, in Course of Construction.
REINFORCED CONCRETE IN SAN FRANCISCO AND VICINITY.

It is beyond question that our State Board of Harbour Commissioners are being guided by their experiences and investigations in the selection of reinforced concrete for permanent structures on the water front of San Francisco.

San Francisco.—The city of San Francisco has shown its confidence in reinforced concrete in a very emphatic manner. It is well known, no doubt, to you all, that the great fire of San Francisco was caused by the breaking of the water supply pipes on the marsh just south of the city, thus leaving our fire department without any water to combat the conflagration. Before a re-built city could be made safe, it was patent to all that some device must be used to protect the city in case the supply pipes should again be broken. It was finally decided to build storage cisterns distributed throughout the city at proper intervals. There have been 80 of these cisterns built in San Francisco, and all of them are constructed of reinforced concrete.

In the new sewer system, for which bonds have been voted in the sum of $5,000,000, reinforced concrete is being used exclusively where the sewers are of sufficient size to warrant it.

The County Infirmary building was built of reinforced concrete, and in such steel frames as have been constructed by the city it is used exclusively for the fire-proofing and floor constructions.



Reinforced Concrete Bridge across the San Joaquin River, California.
REINFORCED CONCRETE IN SAN FRANCISCO AND VICINITY.

Vicinity.—The interior cities of Sacramento, Stockton, San José, and Santa Rosa have each contributed several structures in reinforced concrete, and in each case these are of particular importance in their respective cities. Of the above-mentioned cities, San José and Santa Rosa were severe sufferers from the earthquake and fire. In the city of Oakland are several very large concrete buildings, all built since April, 1906. The City Council of Oakland have just awarded the contract for the construction of an all reinforced concrete wharf.

Bridges.—Previous to 1906 and within the territory included in this discussion, there was but one reinforced concrete bridge of any magnitude. This one was a structure consisting of ten 75-ft. spans, built by the interior counties for highway purposes. This bridge caused considerable inclination among the county officials to weigh more carefully the merits of concrete bridges. By persistent efforts the concrete men are being able gradually to convince the authorities that the extra cost of concrete bridges over steel is money well expended. The result is that there exists to-day over fifty reinforced concrete county bridges, ranging in magnitude from 35 to 130 ft. spans in a section that four years ago contained but one. There is now being constructed by one of the interior counties a bridge consisting of seven 200-ft. spans.

The county surveyors, who under the laws of the State of California control the designing of bridges for the county, are rapidly becoming advocates of the use of reinforced concrete. The bridges that have been built in the interior, which embrace flat slabs, girders and arches, have done an immense work toward educating the people to understand the possibilities and utility of this material. Two counties of California have, through their Board of Supervisors and by resolutions of the latter, committed themselves to the use of reinforced concrete for bridge purposes wherever possible.

THE "PIKETTY" SYSTEM OF CONSTRUCTION.

We give below particulars of a system of reinforced concrete which, although new to this country, has been used very successfully on the Continent for over twelve years.—ED.

MR. PAUL PIKETTY, formerly of the Ecole Polytechnique (the State Training College in Paris for Engineers), over twelve years ago founded a new method of reinforced concrete construction known as the "Piketty" system.

This system is well known on the Continent, and, owing to the development of the business and the success of his method, Mr. Piketty decided to introduce it into the United Kingdom. The head office for this country has been opened at 14 to 18, Bloomsbury Street, London, W.C., from which the firm of Paul Piketty and Co. is transacting business.

In the "Piketty" system a double reinforcement is employed consisting of round steel bars for resistance to tensile and compressive stresses, and of U-shaped loops for resisting shearing stresses. These loops are placed at varying angles. They are inclined at 30 degrees near the supports, then straightened progressively to take the vertical position towards the middle of the beam. Should important shearing stress

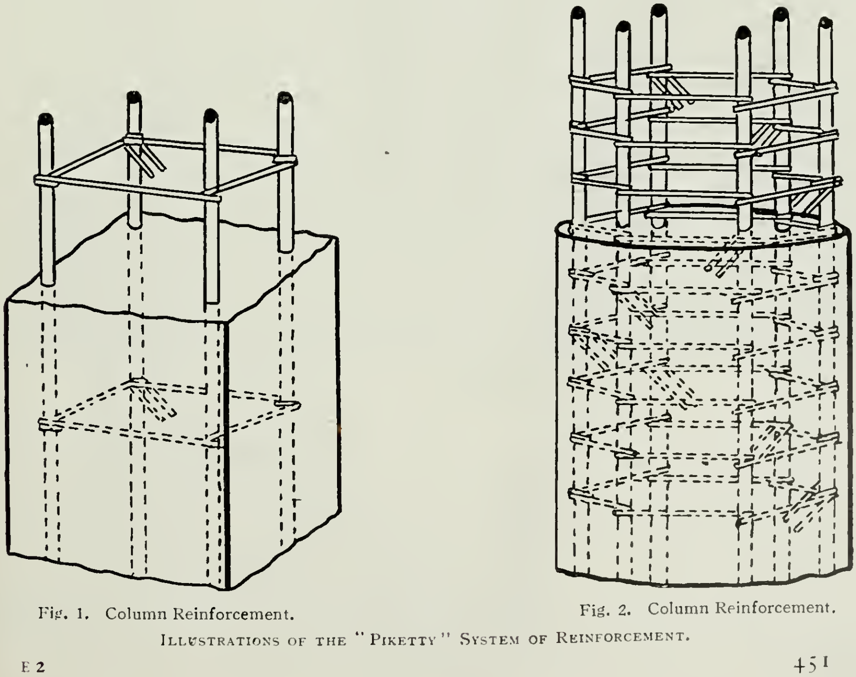


Fig. 1. Column Reinforcement.

Fig. 2. Column Reinforcement.

ILLUSTRATIONS OF THE "PIKETTY" SYSTEM OF REINFORCEMENT.

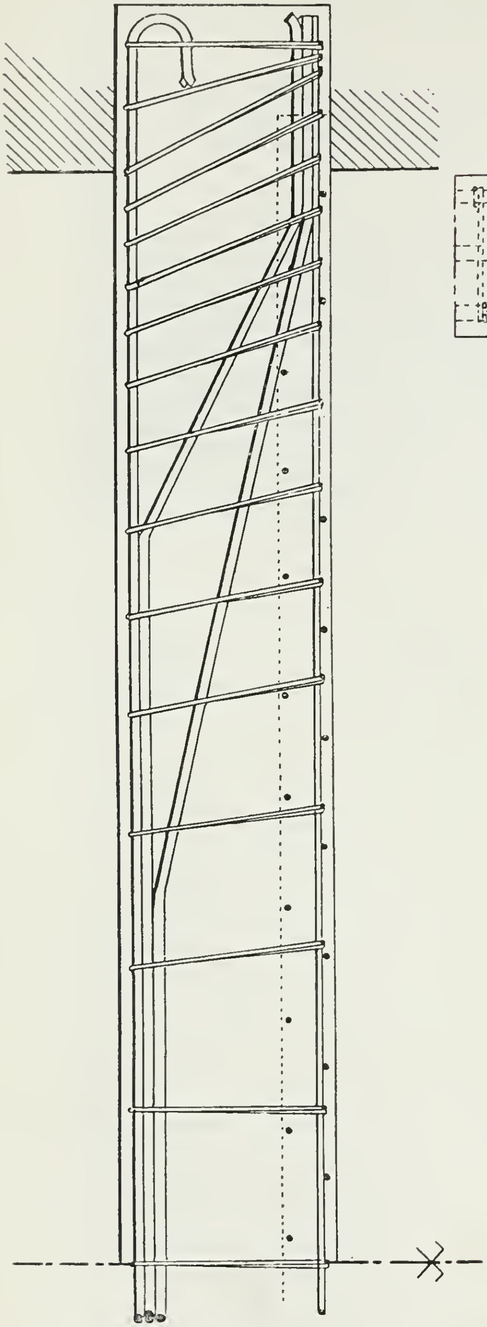


Fig. 3. Reinforcement of Beams.

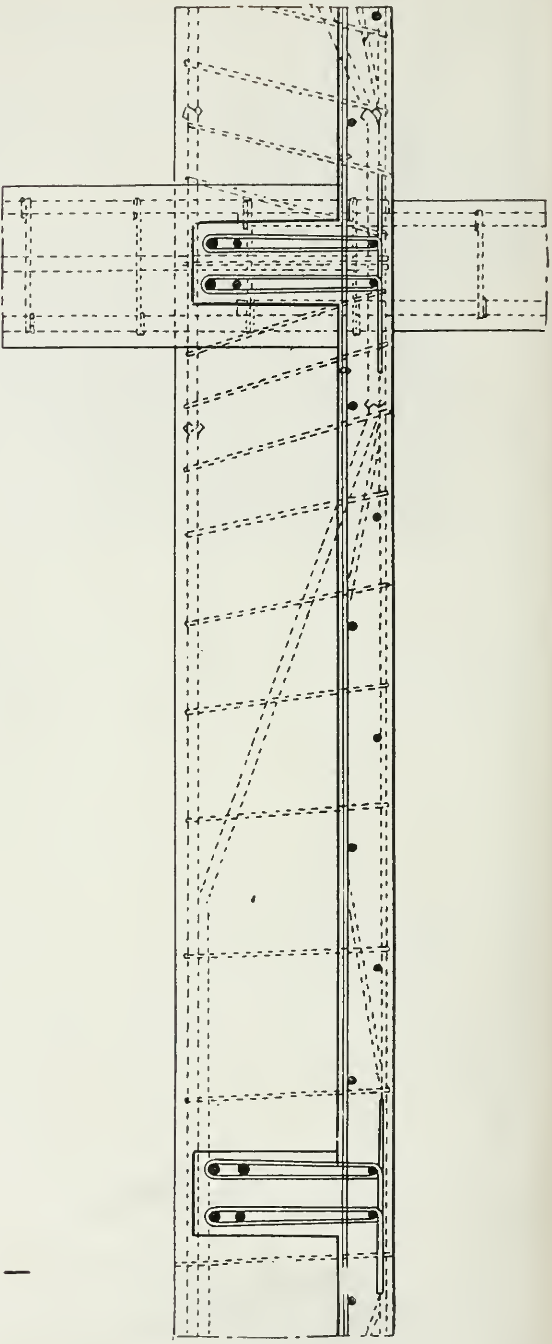


Fig. 4. Reinforcement in Continuous Beams and Slabs.

Illustrations of the "Piketty" System.

be met with, the problem of reinforcing according to tensile directions is still more exacting, and in the "Piketty" system a second disposition is adopted which consists

Reinforced Concrete Viaduct carrying a Railway.

Grand Stand at De Vilieurbanne Racecourse.
Examples of Work on the "Piketty" System.

in adding strong loops inclined at 30 degrees on the horizontal to vertical loops. The beam is therefore reinforced in two directions. The inclined loops or stirrups are fixed around the bars of the lower member and locked to the upper reinforcement.

The slabs in this system are also composed of two reinforcements; the bars of each one are generally laid at right angles to one another.

The columns may be square, polygonal or circular in cross-section, the reinforcement consisting of longitudinal bars braced at intervals by transverse ties, each composed of a steel rod bent around each longitudinal bar and the ends turned inwards, so as to be embedded in the concrete core of the column.

Fig. 1 shows an ordinary square pillar or column, with four lines of vertical reinforcement. *Fig.* 2 is a round column with six lines of vertical reinforcement; laterals are provided in greater number than in *Fig.* 1 in order to raise the ultimate strength by preventing the swelling of the concrete and increasing the compressive resistance of the same. The number of these laterals is determined by calculation.

Colonnade at Contrexville.
EXAMPLES OF WORK ON THE "PIKETTY" SYSTEM.

In *Fig.* 3 is shown a beam with three series of bars—namely, straight bars, bars bent up at a third of their length, and bars bent up at a sixth of their length.

When there is a limitation as regards the depth of a beam, or when it is necessary to set three layers of bars in tension, the middle layer is bent up at a sixth of its length, so that the reinforcement may be better embedded in the concrete.

Fig. 4 shows an assemblage of the members or pieces composing the floor—viz., pillar, beam, small beams, and slab.

The principal advantages claimed for the "Piketty" system are:—

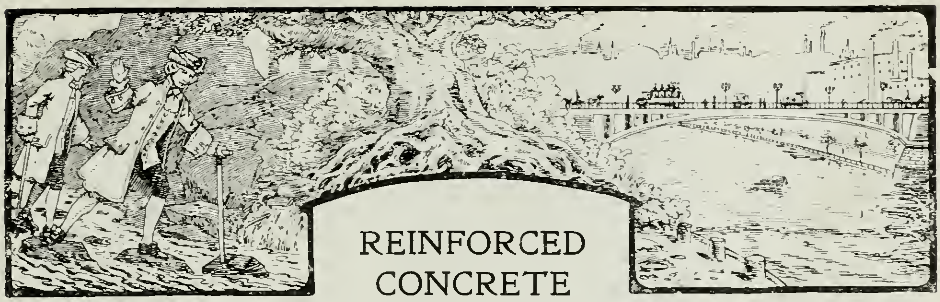
(1) That the sloping portions of the upward bent bars in beams furnish the greater part of the resistance needed against stresses due to negative bending moments near the supports of built-in and continuous beams, the remaining resistance being afforded by the upper straight bars, around which the ends of the stirrups are turned.

(2) That the angular disposition of the stirrups, varied according to calculation in each case, is one offering the most effective resistance to tension on diagonal planes in the concrete.

(3) That the transverse ties in column reinforcement have the effect of bracing the longitudinal bars in their correct positions and of raising the ultimate strength by preventing the swelling of the concrete under compression.

Numerous important architectural and civil engineering works have been carried out by this firm in Europe and abroad. Several works are projected and in progress in this country. Their contractors have been specially selected as having experience in the successful execution of contracts for Government Departments and leading architects, so that those inviting plans may rely on having their work carried out by firms of repute.

Messrs. Paul Piketty and Co. invite correspondence on all matters relating to reinforced concrete. They have just issued an illustrated handbook which gives every information respecting their system and method of working.



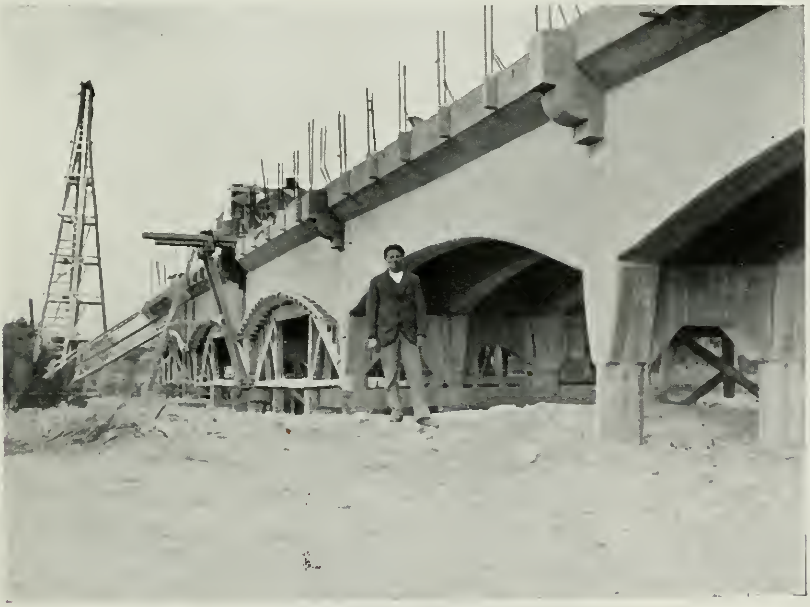
REINFORCED CONCRETE BRIDGE OVER THE ASHLEY RIVER, NEW ZEALAND.

By E. R. MATTHEWS, A.M.Inst.C.E.

As a further instance of the headway which reinforced concrete is making in our over-seas possessions, we are pleased to be able to give an example of a large bridge which has recently been erected in New Zealand.—ED.

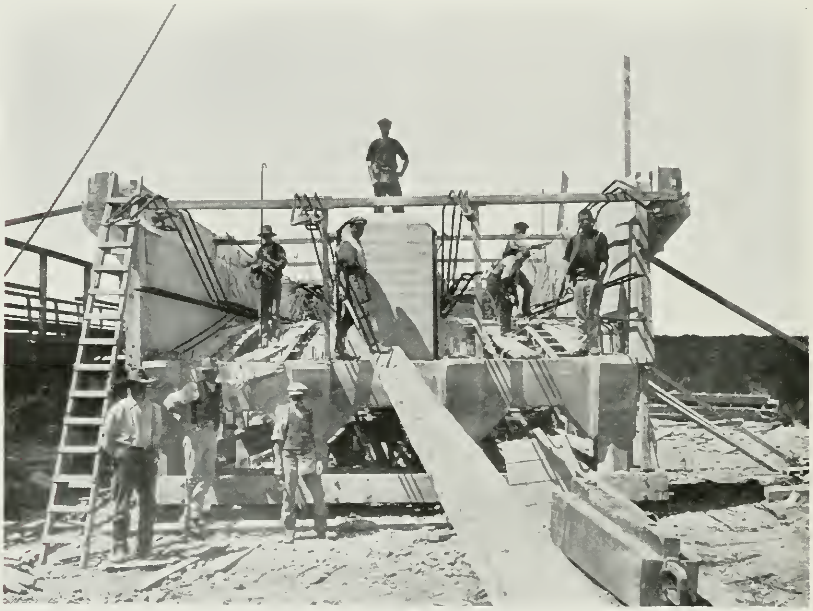
One of the best examples of the use of reinforced concrete in engineering construction in New Zealand is undoubtedly the large bridge which has recently been erected across the Ashley River, near Rangiora, for the Ashley Road Board.

This bridge was built for the purpose of taking the place of an existing timber bridge on the Main North Road. It is 1,025 ft. long, measured between the centres of the piles of the end spans, and has twenty-seven piers, four piles being driven to support each pier, thus making a total of 108 piles. Eighty-four of these are 28 ft. long, and twenty-four of them 32 ft. long.



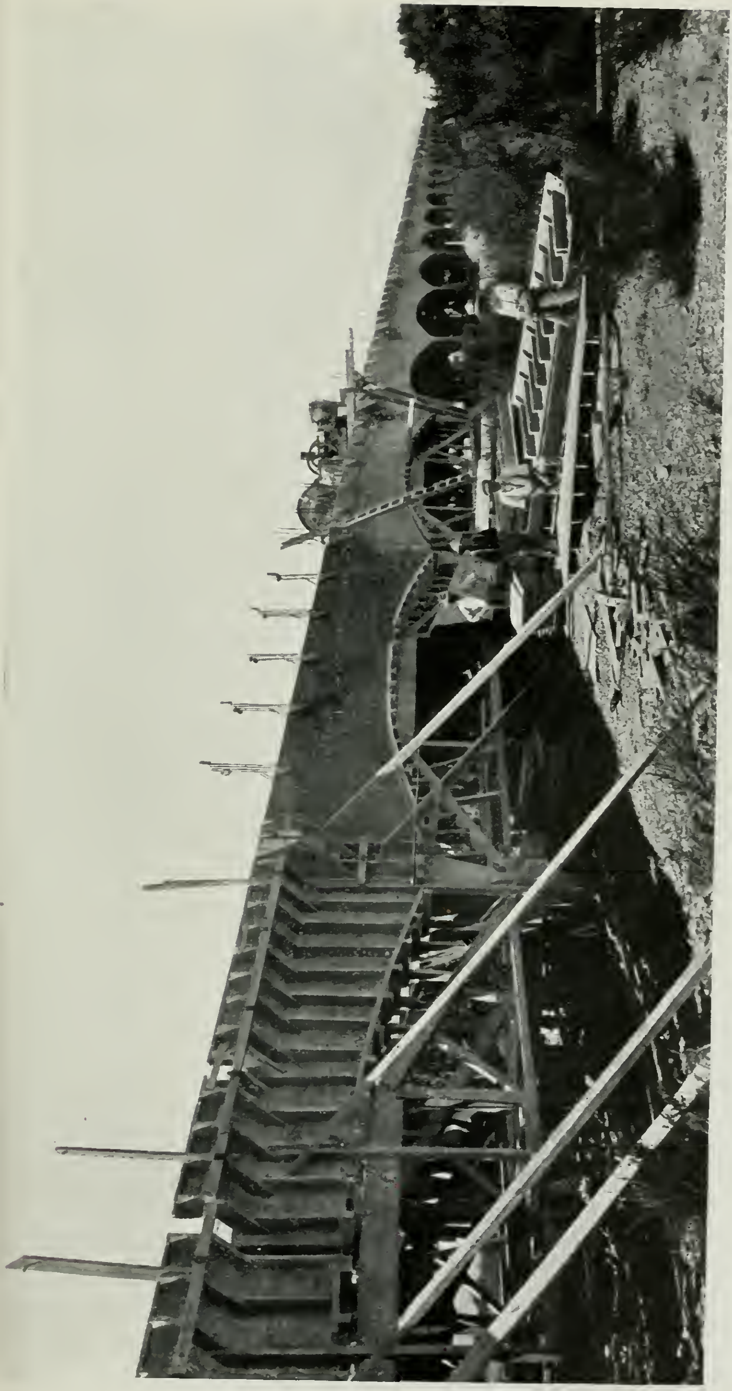
Bridge in course of construction.
Reinforced Concrete Bridge over the Ashley River, New Zealand.

Piling. — The piles are octagonal in shape, and the reinforcement of each consists of eight 1-in. diameter steel vertical rods, bound with looped hoopings and sectionised helical wrappings, these being the patent of the British Reinforced Concrete Engineering Co. Ltd. Each pile was shod with a steel-pointed, wrought-iron shoe, and in the top and bottom 3 ft. of the piles a larger proportion of cement was used in forming the concrete. The heads of the piles were protected during driving, and a 4-ton monkey was used. They were driven from 20 to 25 ft. into the shingle bed of the river, because during floods the swift stream scours its bed from 7 to 10 ft. in depth. A record was kept of the number of blows required in driving each pile.



Forms and steel reinforcement in place.
Reinforced Concrete Bridge over the Ashley River, New Zealand.

Concrete. — The cement used in the work satisfied the tests and requirements set out in the Standard Specification for Portland Cement compiled by the British Engineering Standards Committee. The shingle used in the concrete work of this bridge was broken to the required size by a stone-crusher, the aggregate consisting only of good hard stone, which had been taken from such positions in the river bed as the engineer had directed. The stone chips that were used were also screened, so that the aggregate consisted of stones ranging from $\frac{1}{4}$ in. to 1 in. The sand used was coarse, sharp, grey sand, capable of passing through a sieve having holes of 1-10 in. diameter, but being retained on one having holes of 1-25 in. diameter. The proportions of the concrete were as follows :—*Piles.*—One part cement, $2\frac{1}{2}$ parts stone chips ($\frac{1}{4}$ in.



Side view during erection.

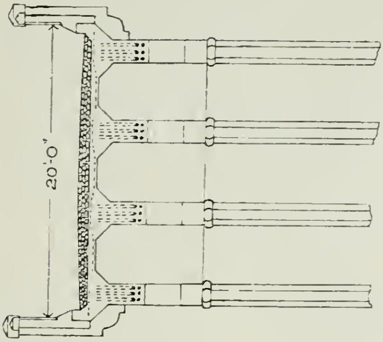
Reinforced Concrete Bridge over the Ashley River, New Zealand.

to ½ in.), and 1½ parts sand; remainder of work: 1 part cement, 1 part stone chips (¼ in. to ½ in.), 3 parts broken stone not exceeding 1 in., and 2 parts of sand. The stone crushing plant had a proper rotary screen for the sizes specified, the jaw plates of the crusher being fitted with Hadfield's (Sheffield) Special Hard Era Manganese steel. A strong blast of air was driven by a fan through the broken stone as the latter dropped from the jaws of the crusher, so that all dust was entirely removed; in fact, the

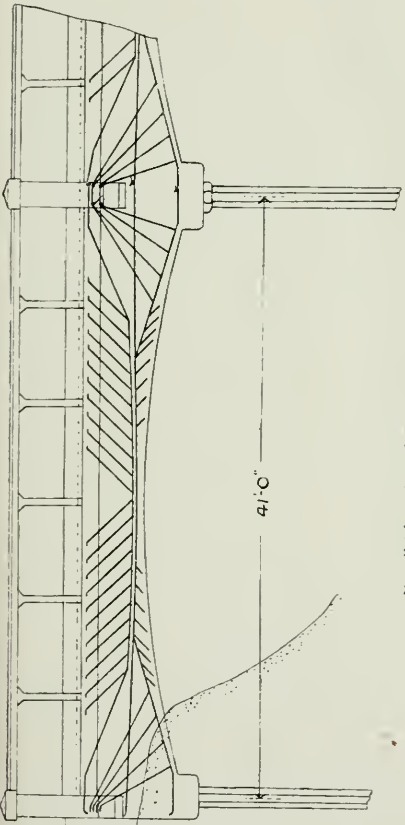
View along roadway before laying road surface.

Reinforced Concrete Bridge over the Ashley River, New Zealand.

dust was driven through a timber trunk 18 ft. in length, so as to entirely remove it from the neighbourhood of the crusher. Every arris in the concrete received a 1-in. chamfer, and the forms were well wetted before the concrete was deposited. Great care was taken when jointing new concrete to that which had been previously set, the face of the latter being well cleaned down, hacked and grouted, the latter, consisting of 1 of cement to $\frac{1}{2}$ of sand, laid on to the consistency of thick cream. After these precautions had been taken the new work was proceeded with. Immediately the concrete had been laid it was covered up with sacks which had been well wetted, so as to prevent the sun or wind drying the surface of the concrete; after it had well set the concrete was watered every two hours for a period of four days. During the frosty weather no concreting was proceeded with after 4 o'clock in the afternoon. It was also well covered up with sacks to prevent damage.



Cross Section through Centre of Span.



Detail Elevation of End Bay.

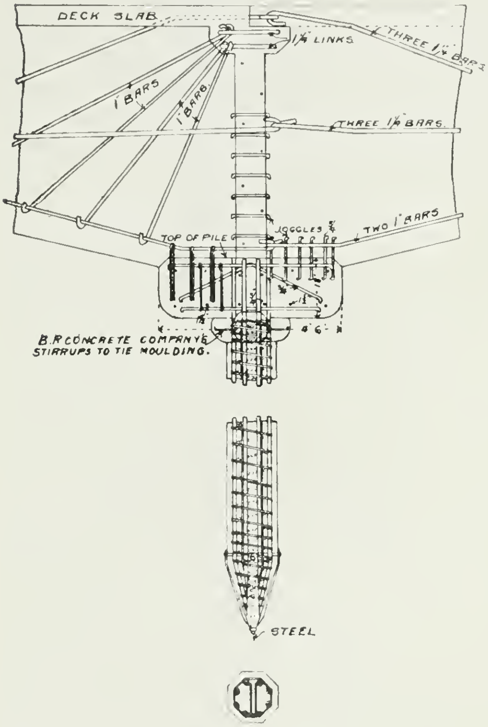
Reinforced Concrete Bridge over the Ashley River, New Zealand.

Timber.—The timber forms were so constructed as to be absolutely rigid; they were well braced and fixed on unyielding supports. The planking consisted of 4 in. by 1½ in. white pine, bevel edged, wrought smooth, straight and flat on the concrete side. It was well soaked with raw linseed oil on the plain side, and was kept thoroughly soaked with kerosene and soap mixture throughout the time that it was in use, so as to prevent the adhesion of the concrete. Studs for the support of the planking were not less than 4 in. by 3 in., and spaced not more than 2 ft. 6 in. apart.

Reinforcements.—The ultimate strength of the steel was specified to be not less than 58,000 lb. per sq. in., nor more than 66,000 lb.; the elongation to be not less than 20 per cent. in a length of 8 in. of test bar. The elastic limit to be not less than half of the ultimate strength. These tests were made in England, and the steel in every case came up to the requirements. In placing the reinforcements, wires were used to tie up the reinforcement in its exact position. All bars were hooped together besides being bound with No. 14 wire to prevent the hooks from springing out. Where bars crossed each other they were lashed together by two turns of No. 14 wire. Paragon stirrups were used throughout. The bars were to be well washed with caustic soda solution, if necessary, before being placed in position, so as to remove any dirt, paint, or grease. They were also well brushed down with wire brushes, and the concrete was worked in around the bars, special tools being employed for the purpose. All ends of reinforcing bars were cranked 2 in., except where hooked, the metal being bent at red heat only.

Removal of Forms.—No forms were removed until a period of fourteen days had elapsed since the setting of the concrete. The girders were rigidly supported for twenty-eight days. All concrete work was stopped square over the centre of a support and formed into a series of steps, so that there would be a proper connection and bond between the new and the set work. The horizontal steps were at least 2 ft. wide, and the vertical ones 1 ft. high.

Finishing of Concrete Surfaces.—The finished surfaces of the concrete were not plastered, but as great care had been taken in the erection of the forms, and these had been so close jointed, a perfectly even and smooth face was given to the finished concrete work without the need of plastering. It will be remembered that this is the American method of completing the face of concrete structures, and one which the author has from time to time advocated in this country, especially in regard to the finishing of reservoir walls and bottoms. The plastering of such work he deprecates. When the face of the work had been properly finished it was wetted and coated over with a thin coat of cement grout, applied with a white-wash brush. Expansion joints were left in the concrete where shown in the accompanying illustration.



Detail of Pile.

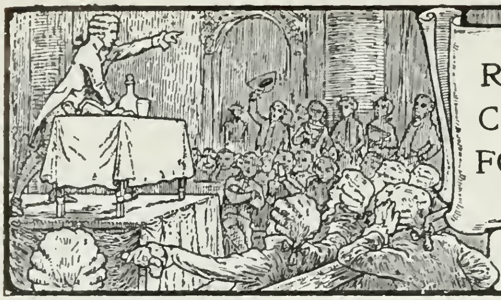
Reinforced Concrete Bridge over the Ashley River, New Zealand.

Roadway of Bridge.—The length of this roadway is 1,027 ft., and the construction of same is shown on page 458; 2½ in. broken stone was used for the metalling of the road, and the width of the latter is 20 ft.

Test of Bridge.—The Government having made a special grant in aid of the construction of this bridge, required that it should be tested by driving two traction engines *side* by *side* over the bridge. The collective weight of these was 30 tons, and the maximum deflection in the centre of each span under this test load barely reached one-fiftieth of an inch.

Conclusion.—The bridge described in this article is, with the exception of the fine arched Grafton Bridge, Auckland, the largest reinforced concrete bridge in New Zealand. It was designed by and erected under the supervision of Mr. F. W. Marchant, M.Inst.C.E., of Timaru, New Zealand, the reinforcements being supplied by the British Reinforced Concrete Engineering Co., Ltd., of Manchester, and reflects great credit upon them both as regards design and construction.

Another reinforced concrete bridge built by the same engineer has a point of special interest. This is a bridge, 960 ft. long by 20 ft. wide, in 40 ft. spans, built across the Opihi River, S. Canterbury, New Zealand. It has no expansion joint in the centre. Between day and night this bridge lengthens and contracts $\frac{3}{4}$ in., due to change of temperature. There is no crack nor symptom of one anywhere in it.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE INSTITUTION OF WATER ENGINEERS.

PORTLAND CEMENT ON PUBLIC WORKS.

Paper by GERALD THORNHILL EDWARDS, Assoc.M.Inst.C.E.

The following is an abstract of a paper recently read by Mr. Gerald Thornhill Edwards, A.M.Inst.C.E., before a meeting of the Institution of Water Engineers.

Civil engineering in all its branches is at the present time so dependent on the use of Portland cement, that a thorough knowledge of the qualities of this sensitive material is absolutely essential to a practical engineer.

During the last few years there have been many improvements in the manufacture of Portland cement, the most notable being the introduction of rotary kiln burning and fine grinding. These improvements, in addition to cheapening the production, and consequently widening the demand, have introduced new problems for both manufacturer and user.

It is the author's intention, in this paper, to enumerate the different tests which should be carried out in order to ensure only perfectly sound cement being used; to explain why such tests are necessary; and, further, to comment generally on the properties and use of Portland cement.

Laboratory on Works.—A cement-testing laboratory should consist preferably of three rooms. One of these must be so constructed that its temperature can, by means of some simple contrivance such as an oil stove, be so regulated that it never falls below 58 deg. nor rises above 64 deg. Fahrenheit. This room must be absolutely free from all draughts. In it there should be a slate slab on which the cement and water can be mixed, rows of shelves where briquettes and pats may be kept in glass dishes, damp boxes in which newly made pats may be placed, and the initial set testing instrument.

The second room is a small one, and used as an office for writing up the results of tests, checking cement consignments, etc.

The third room should be set apart for carrying out the boiling and all mechanical tests, with the exception of the initial set test, and should contain all the various machines and appliances required for the purpose.

Chemical Analysis.—A representative sample from each parcel of, say, fifty or 100 tons should be analysed, and the result compared with the specified analysis.

The usual proportions of the principal component parts of Portland cement are lime, 62 per cent. to 65 per cent.; silica, 18 per cent. to 20 per cent.; and alumina, 6 per cent. to 8 per cent.

The analysis should not show more than 3 per cent. of magnesia, 2·75 per cent. of sulphuric anhydride, and 1 per cent. of insoluble residue.

The principal development of recent years has been the increased percentage of lime in the better class cement. A few years ago 60 per cent. was considered the outside limit, while it is quite common nowadays to find sound cement containing 64 per cent. of lime.

Care should be taken, however, that no more lime is contained than will combine with the silica and alumina present, and the chemical formula for this, given in the British standard specification, should be strictly complied with, otherwise free lime will be found in the product.

The specific gravity should be not more than 3·25 and not less than 3·15 when the cement is freshly ground.

Fineness.—Modern engineers specify a degree of fineness that would have filled with dismay the manufacturers of twenty years ago.

At that time the usual fineness test was that the cement should pass through a 76 by 76 sieve—*i.e.*, one having 5,776 meshes to the square inch—with not more than 10 per cent. residue. At the present time it is quite common to specify not more than 10 per cent. residue on a sieve having 32,400 meshes to the square inch.

Initial Set.—To conduct this test, neat cement is mixed with as much water as will ensure a standard consistency, which can be gauged by a standard plunger; the mixture being filled into a mould resting on glass and then placed in the constant temperature room in a position free from draughts.

From time to time the pat must be placed under the needle used for testing the initial set. The British standard specification fixes the initial set as taking place when the needle fails to penetrate to the botom of the mould. This is not correct in the case of some slow-setting cements, which have a kind of false initial set, as they resist the needle at about 1 millimetre above the bottom of the mould and remain in that condition for some time, and in some cases for an hour or two.

Now, the larger the mould, the nearer the approach to working conditions. Unfortunately, however, the size of mould is limited by the amount of quick-setting cement that a man can handle. It has been found in practice that the best size of mould is 1½ in. deep, 4⅛ in. diameter at the top, and 4 in. diameter at the bottom, made of ½-in. iron.

Before commencing this test, care should be taken that the air, mould, water, and cement are at the same temperature—between 58 deg. and 64 deg.

The sample must be exposed in the constant temperature room for twenty-four hours before testing.

The Vicat needle is the instrument generally used for obtaining the initial set; but some modification of it is undoubtedly necessary, as, owing to the fact that the needle has to be lowered by hand, the personal element is bound to creep into the result. This difficulty is completely overcome by the simple contrivance of attaching a string to the top of the needle and leading it over a couple of pulleys to a small reel. By turning the handle of this reel the needle can be lowered at the same rate and with exactly the same force every time.

The initial set of any consignment of cement should be regulated according to the conditions under which the cement is to be used. Thus, if an engineer is putting in concrete under water he should order quick-setting cement, and if he is building a dam where the wet concrete is shovelled about, punned, and trampled upon for an hour after mixing, he should order a slow-setting cement. In other words, the time between the gauging and initial set must be longer than the interval necessary to allow of the concrete being placed in its final position.

Shrinkage.—It will be noticed that the mould referred to above is tapered, being 4⅛ in. diameter at the top and 4 in. diameter at the bottom. The reason for the taper is that, if made so, a test for shrinkage may be carried out with little trouble.

There is no shrinkage test mentioned in the British standard specification, but it is important, as shrinkage often occurs, and, if not detected, is liable to produce contraction cracks in concrete.

It is readily cured by a few days' aeration—a fact which denotes that the fault is due simply to immaturity of the cement.

Final Set.—It is only necessary to touch lightly on the question of final set, because it is of very little practical use, and also because the method of conducting the test is unscientific and arbitrary. The final set is said to take place when the needle fails to make an impression on the pat, but a mark which is perceptible to one person may be quite imperceptible to another.

A far better test is to ascertain the pressure necessary to penetrate with a standard drill a standard distance into the block. By this method the time necessary for any cement to obtain a standard degree of hardness can be ascertained.

Soundness Tests. — The soundness tests are best conducted in the following manner:—

Take 8 ounces of cement, mix with 26 per cent. of water (rather more if quick-setting), work up quickly into a stiff paste and divide into four equal portions. Place the first three on pieces of glass and work up with a trowel into smooth round pats, leaving them about half an inch thick. Put one, the "air" pat, in a place free from draughts and of even temperature; the next, the "boiling" pat, into a damp box or humid chamber for 48 hours. Allow the third, called the "water" pat, to remain in the air until a skin has formed, then place it in water of a temperature not less than 58 deg. and not more than 64 deg. Fahrenheit.

Take the remaining portion of cement and, with the addition of a few drops of water, work it up and fill a Chatelier mould, then, with glasses on the top and bottom, weight it and place in water of the same temperature as the above for 48 hours, afterwards boiling it for six hours with the "boiling" pat before mentioned.

"Air" Pat. — After the expiration of 48 hours examine this carefully, and if it is wrinkled or shows small hair-like cracks, it will be evidence, in the former case, of shrinkage or contraction of the cement, and, in the latter case, of the presence of free lime. Of late years, since the introduction of scientific methods of manufacture, one rarely meets with a cement so lacking in combining qualities as to show cracks in this test; nevertheless, if they are observed, the presence of free lime will be confirmed by the cracking and warping of the "water" pat, by expansion in the "Le Chatelier" test of more than 4 millimetres, and also by signs of disintegration and soddenness in the "boiling" pat.

If the test is not confirmed in this way, it will generally be found that an unobserved current of air, or the overheating of the room has brought about too rapid evaporation, and consequently caused the pat to be defective.

"Water" Pat.—After 48 hours' immersion the pat should be found to be firmly stuck to the glass at the edges and no crack or blemish should be visible.

If small and superficial cracks appear at the top or edges, but the pat adheres firmly to the glass and the other tests are satisfactory, it may be safely assumed that the disturbance owes its origin to an undue proportion of the gypsum present in the sample, being brought to the surface in the process of forming and patting.

Some sound cements will not endure in this test the unbalancing of their constituent parts.

On the other hand, if the pat is cracked through, warped, or lifted from the glass at the edges, there can be no doubt that the sample is immature. The proof of this lies in the fact that such a sample can always be made perfectly sound and able to pass this test by a simple process of aeration.

"Boiling" Test.— If there is perefect combination in setting, this pat should dry almost instantaneously on being taken from the boiling water, and have a sharp metallic ring when struck.

On the contrary, if the pat has a sodden appearance or a network pattern shows as the water dries off, or, worse still, if obvious cracks appear, it is a sure sign of immaturity and consequently of a lack of perfect combination.

Le Chatelier Test. — If the distance between the points of the mould when measured after boiling is more than 4 millimetres greater than the measurement before boiling, the cement is probably immature, and this will generally be corroborated by the water and boiling tests.

If the expansion is only one millimetre or less, it will usually be found that there is perceptible shrinkage in the cement, and confirmation of this must be sought in the tapered ring mould test.

Tensile Tests. — Briquettes for tensile tests are made in two forms—namely, (1) neat briquettes (*i.e.*, neat cement mixed with water only), and (2) sand briquettes, made of one part of cement and three parts of standard sand.

The advent of fine grinding has been the death of the neat tensile test, for it has been established that neat briquettes made from very finely ground cement are weaker at certain ages than the same cement coarsely ground. Such a conclusion is very misleading, and the sand compression tests prove that the neat tensile tests are inaccurate.

Now, with regard to the sand tensile test, it has been found by experiments that the personal element enters very largely into the making of briquettes, so that there is great difference between the results obtained.

Moreover, it is impossible to standardise this test, for each sample requires a different percentage of water, and only an expert can tell how much water should be used for any particular cement. Any increase in this amount of water brings about a marked decrease in the initial strength of the hardened specimen.

Taking all these facts into consideration, how can it be argued that the tensile test gives one any idea of the compressive strength of concrete?

More important than this, however, is the fact that the tensile test altogether fails to reveal a defect that unfortunately appears occasionally in some of the modern cements—namely, the hardening of the outside skin while the interior of the cement remains quite soft. A tensile briquette has a periphery of 4 in., while its cross section is only 1 sq. in.; so that, assuming each of the four surfaces hardens, the strength of the briquette will be almost as high as if it had hardened right through.

A cement may, therefore, pass the British standard specification tests, although it has this very serious defect, and it would seem that the only method of detecting it is to bore through a pat, measuring the penetration and the resistance.

This constitutes a test for hardening, which can be conducted after either 24 or 48 hours. The test, being made on a small neat cement pat, is very much accelerated, and the result is comparable with the state of the concrete at about a week old.

Apparently, then, the only reason why the tensile test should not be superseded by a hardening test is that no standard hardening strength has yet been established; but that is only a minor matter and can easily be remedied in a very short time.

Hardening.—A test of the initial hardening of Portland cement is urgently needed and would be very useful for the following reasons:—

(1) It would take the place of the final set test.
(2) It would reject defective cement which hardens only on the surfaces.
(3) It would give a better idea of the comparative strength of the sample than the tensile test does.
(4) It would enable a user to differentiate between quick and slow hardening cements; and
(5) It could be standardised.

Reasons (1), (2) and (3) have already been referred to.

Coming to the fourth reason, it may, perhaps, be as well to state first that *setting* and *hardening* are not synonymous terms. The two properties are totally different, and seem, in point of time at any rate, to have no connection with each other. There is, however, a sort of general law, which has been established by numerous experiments, that given two samples of the same make of cement, one quick setting and the other slow setting, the quick setting is the slower hardening.

For structural concrete work over centres, between shutters, and in deep heavy trenches, a quick-hardening cement is essential. In placing a contract for such work, the usual practice hitherto has been to order a quick-setting cement, under the impression that this would ensure quick hardening. But this has been proved above to be quite an erroneous idea, although some quick-setting cements may give good hardening results.

It is impossible to know whether a cement hardens quickly or slowly, unless it undergoes a hardening test.

Passing to the fifth reason, when the size of the initial-set pat has been fixed, that set will be standardised. The pat used in ascertaining the initial set can also be used for the hardening test.

Aeration.—The question whether cement should or should not be aerated is always a bone of contention on big public works. Up to quite recently, most engineers specified that all their cement should be aerated.

Manufacturers, nowadays, are willing and eager to guarantee their particular product as ready for immediate use; but, even in the best brands, one often comes across some little defect that is readily cured by a few days' aeration.

Apart, however, from any advantage gained in the quality of a cement by aeration, comes the question, Is it economical? It is, of course, a well-established fact that aeration increases the bulk of dry cement; but what may not be so well known is, that cement also increases in weight during aeration, through absorption of moisture from the atmosphere.

Further, aerated cement requires more water to combine properly than non-aerated.

Experiments have been carried out to ascertain the degree of increase in weight and bulk, and these tests show that, due to aeration only, the increase in weight is 3 per cent., in bulk 11 per cent., and in bulk of neat cement mortar $8\frac{1}{2}$ per cent.

By the courtesy of Mr. G. N. Yourdi, M.Inst.C.E., the author is enabled to state in relation to the aerating shed of the Birmingham Corporation, in the Elan Valley, that there were booked out of the shed about 2,000 tons more than were received into it, therefore, on paper, the gain in passage through the shed was, say, 2,000 tons, or $1\frac{2}{3}$ per cent.

This apparent gain may or may not have been wholly due to aeration. The bags sent out of the shed were supposed to be full weight, and liable to be checked at any time by one of the engineers. Then, again, considerable waste often occurs in an aerating shed, due to the flour being carried away by the draughts, or on men's clothes and railway wagons.

Measurement of Cement when making Concrete. — The author cannot too strongly deprecate the existing practice of measuring cement by volume instead of by weight. It affords an astute contractor a splendid opportunity of making a handsome and perfectly legitimate profit at the expense of the quality of the work.

The usual practice in this country is to specify, in regard to concrete, that it shall consist of certain proportions of cement, sand and broken stone, and that proper gauge boxes for measuring these proportions are to be furnished; in other words, the cement is to be measured by volume.

Such a clause is a source of joy to an experienced contractor, because he knows that, by careful management, he can make 90 to 95 tons go as far as the 100 tons of an inexperienced man.

This saving is, of course, effected at the expense of the quality of the work.

It is difficult to see any reason for continuing the present practice of measuring cement by volume, especially when it is so much easier and simpler to measure by weight. The manufacturer weighs the cement and is prepared to guarantee that each bag weighs so many pounds, therefore each bag contains a known quantity and does not need to be reweighed.

If engineers would specify that concrete shall consist of certain proportions of cement, sand and stone, the cement to be measured by weight and gauged with the sand and stone on the basis that one cubic foot of cement weighs 90 lb., the contractor could easily make his ballast box or hopper to suit either one or two bags of cement and no measurement or weighing would be necessary.

THE DISCUSSION.

The President (***Mr. Edward Sandeman***) said he was afraid it was rather difficult for anyone who had not had to deal with large masses of cement to discuss the paper. There were many interesting points in it; and one of the most interesting referred to the matter of quick-setting cement being a slow-hardening one. This was really a remarkable feature, because it had been the accepted view that a quick-setting cement was a quick-hardening one. It was quite an anomaly. He (the President) thought it would be found in practice that, as Mr. Edwards said, if one wanted a cement for use in tunnelling work, where concrete was employed to support the roof, a quick-setting cement should not be used, but a slow-setting cement which was quicker in hardening, because what was wanted was immediate strength. There was one interesting experiment Mr. Edwards had not referred to. It was made some time ago; and it showed in what way they could find out interesting matters connected with cement by observation even on a small scale. He referred to the initial set of cement. On his own works they had had several batches of cement which varied very much. With rotary cement it was found that the initial setting point was almost immediate. It set almost as soon as it was mixed with water; whereas the old chamber cement took about ten minutes ordinarily when new. They had some cement which had an initial set of three minutes and some which had an initial set of about fifteen minutes. It struck him as an interesting point to be tested, what would happen if one used together these two cements with different initial sets. An experiment was made by putting half of the one and half of the other together; and it produced a cement which set in nine minutes. So far as could be observed, there seemed to be no disadvantage in mixing the cements with different initial sets of this sort. Since the introduction of rotary cement there had been several changes in the behaviour of cement. In the first place, the initial set, as he had pointed out, had become quicker; and it was necessary now, if new rotary cement was used, either that it should have been dealt with by gypsum, thus lengthening the set, or that it should have been hydrated, or that the clinker should be hydrated.

Dr. H. Lapworth (Derwent Valley Works) remarked that most of them had found the old cement tests were unsatisfactory; but no one had been able to remedy them. The late Dr. Deacon did not believe in tension tests; and he had a very elaborate compression test which gave satisfactory results. The hardening tests described by Mr. Edwards were new; and he (Dr. Lapworth) believed in them.

Mr. J. S. Pickering (Cheltenham) said the author had referred to shrinkage tests. His suggeston of the taper ring to ascertain the shrinkage was novel, so far as he (Mr. Pickering) was concerned. It seemed to him, however, to be a simple and effective method. The various suggestions in the Paper were very valuable, particularly those relating to the hardening tests. They all found that some such test as that was very desirable. He did not, however, agree with the author that one could get almost an equally good tensile test by merely hardening the periphery of a briquette as one would by a thorough hardening. The tensile strain must depend upon the consistency of the hardening throughout. He thought the time would come when it would be felt that the cement which was supplied from the manufacturers by the latest process of manufacture might not necessitate aeration. He did not think it necessary to advocate aeration merely on economical grounds, except in very large works; and even there he did not believe it was worth all the labour put into it. He also thought it an important point in mixing concrete to specify the weight of the cement instead of the volume. It was a practice they ought all to adopt. The present method of specifying the proportions by volume should be changed to weight.

Mr. Ashton Hill (South Staffordshire Water-Works Company) alluded to the drills used by the author, inquiring how he found them answer. Cement was not a nice material to drill; and it seemed to him, if he drilled many samples with the same drill he would get different results with only the same weight on it.

Mr. Edwards said they had had a drill working every day on about six samples for over three months; and the difference was very slight.

THE INSTITUTION OF CIVIL ENGINEERS.

Below we give summaries of two Papers recently read before the Institution of Civil Engineers.

THE DIRECT EXPERIMENTAL DETERMINATION OF THE STRESSES IN THE STEEL AND IN THE CONCRETE OF REINFORCED CONCRETE COLUMNS.

By W. C. POPPLEWELL, Assoc.M.Inst.C.E.

IN carrying out the experiments described in this Paper the author sought for a satisfactory method of measuring the shortening of the steel bars and the simultaneous shortening of the adjacent concrete under the loads applied to reinforced columns. From these measurements, if they were reliable, he thought it would be possible to calculate the stresses in the steel and concrete, when the elastic moduli for the two materials were known. It was also thought that measurements made in this way would be the means of revealing any movement of the steel relatively to the concrete.

The main experiments were carried out on five columns 6 in. square, each reinforced by four round-steel bars $\frac{3}{4}$ in. in diameter. Loads were applied in a testing-machine, and corresponding shortenings of the steel and concrete were measured by means of Martens extensometers. For the steel these were applied to the ends of pairs of pins projecting from the reinforcing bars through holes in the concrete, and for the concrete they were applied to the surface as near as possible to the steel. From the results of the experiments, four pairs of curves were plotted for each column, one pair for each of the bars. These curves are given in full detail in the Paper and their peculiarities are discussed.

Besides the main experiments, others were carried out to compare the effect of loading when the load was uniformly distributed over the end of the column and when it was applied in the centre. This enabled a comparison to be made between the effect of having the load transmitted directly to the ends of the bars, and having it communicated to the bars through the holding grip of the concrete. The result showed practically no difference. The experiments to find out the value of the modulus for the steel and the concrete yielded values respctively of 30,200,000 lb. and 1,535,000 per sq. in.

A further set of experiments carried out to determine the intensity of the frictional grip of the concrete on the steel resulted in values ranging from 300 to 600 lb. per sq. in. of bar surface, to cause slipping. The stresses in the steel and concrete, calculated for a working-load of 13½ tons which the columns were designed to carry, were found to be, respectively, 437 lb. and 8,650 lb. per sq. in. This gives a load on each bar of 1·7 ton, and, comparing this with the load required to push one of the bars through the concrete, as found from the experiments on frictional grip, it is evident that from this point of view there could not have been any slipping of the steel in the concrete.

The author considers that the manner in which the strain measurements were made proved very satisfactory, and that the method might be extended to other cases. A careful inspection of the plotted diagrams appears to show that, when all the effects of eccentric loading have been eliminated, there is no evidence to indicate that slipping took place; and it is evident that in columns of this kind made up with plain smooth bars the two materials behave like one so far as their strain effects are concerned.

COMPOSITE COLUMNS OF CONCRETE AND STEEL.

By WILLIAM HUBERT BURR, M.Inst.C.E.

The effect of a concrete filling on increasing the carrying-capacity of a steel column having never been sufficiently investigated, the present series of tests were carried out by the author. The columns tested consisted first of two types of built-up columns of plain steel, and secondly of exactly similar steel members filled with concrete. The reinforced concrete columns were filled with 1 : 2 : 4 concrete, and were tested at three months. The steelwork consisted in one case of four vertical steel angle-bars arranged as the four corners of a square and braced together with lattice bars to form a square column 6⅝ in. in exterior dimensions, and in the second case of four vertical channels arranged with their flats forming four opposite sides of an octagon and wrapped at intervals with batten plates bent round in the form of an octagon 7½ in. across the flats. Only the concrete lying within the exterior dimensions of the steelwork was included in the calculations. All the columns were 7 ft. long.

Four of each type were tested, two filled with concrete and two without concrete. The plain steel columns withstood an average total load of 67 tons on an area of 4 sq. in. in the case of the angle construction before failure; and of 68 tons on an area of 4·76 sq. in. in the channel construction, which was not so securely braced.

The addition of concrete increased the maximum loads before failure to an average of 98 tons on a total combined area of 42·25 sq. in. in the angle reinforcement, and to 96 tons and 112 tons respectively on an area of 49·75 sq. in. for the two channel-bar columns.

The modulus of elasticity of the steel being known and the compression of each column under successive loadings being measured, the modulus of elasticity of the concrete and its corresponding stress can be calculated. In the case of the angle-bar columns this modulus was 2,321,000 lb. per sq. in. at a stress of 1,360 lb. per sq. in., but decreased as the stress on the columns increased.

Finally, a description of the method of failure of the various columns is given; also curves showing the lateral deflection and main compression. The author concludes by suggesting that the tests would justify working-stresses as high as 500 lb. to 750 lb. per sq. in. in columns of this nature, and possibly higher stresses for structures of unusual magnitude.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE COAST DEFENCE WORKS IN SUSSEX.

THE work of foreshore protection on the Sussex coast near Brighton has been going on steadily for some years past, the object being to check the coast erosion and to protect the cliffs, which rise to a height of 120 ft. in some parts. At first timber, masonry, and concrete were used, but subsequently reinforced concrete was decided on as the most advantageous manner of carrying out this work.

Five groynes were first constructed at Roedean by the Brighton Intercepting and Outfall Sewers Board, each 500 ft. in length and spaced about 500 ft. apart. On completion of these a contract was placed by the East Sussex County Council for one 500-ft. groyne at Roedean. Following these, the Council placed a further order with Messrs. Holloway Brothers, who had done all the previous work, for two groynes, each 500 ft. long, and 1,000-ft. run of sea walls at Saltdean, and the following additional contract has recently been completed by the same firm near Rottingdean :—Ten groynes, each 500 ft. long and 2,000-ft. run of sea walls in about 200-ft. lengths running at right angles to each groyne, the approximate quantities in the latest contract being 4,500 cu. yds. mass concrete, 84 tons steel reinforcement, and 3,900 cu. yds. excavation in shingle and chalk.

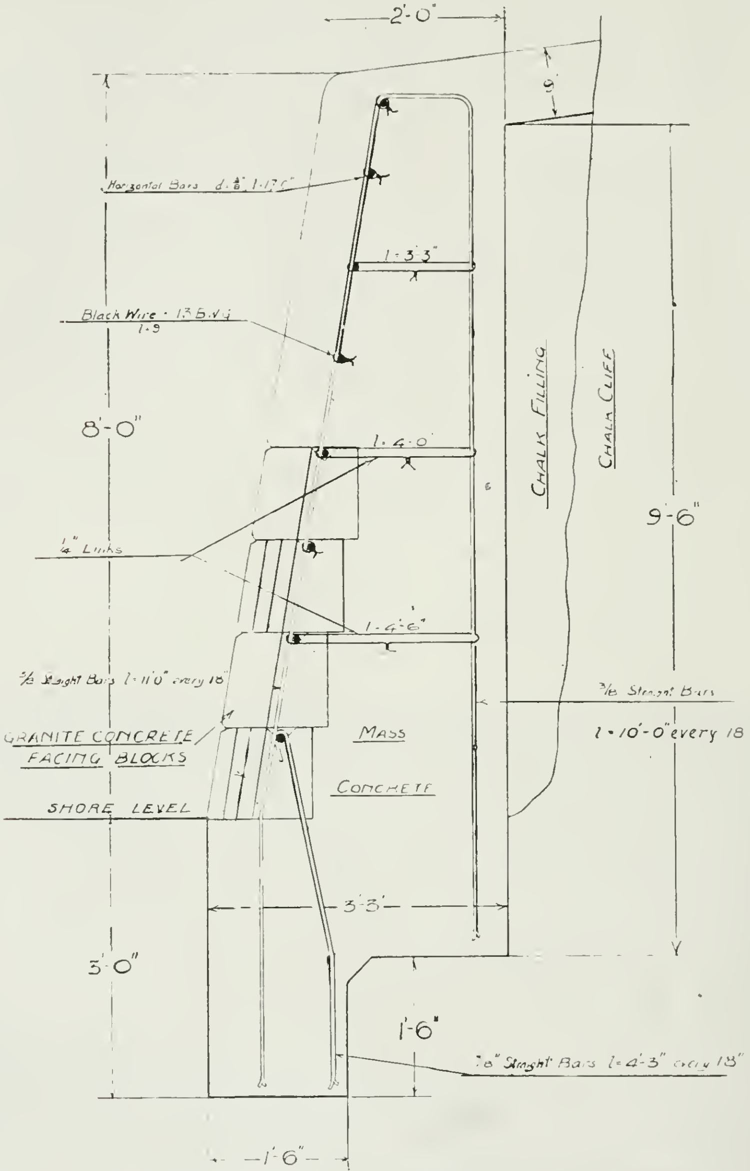
For the purposes of the work the contractors established a depôt on the cliffs, where the concrete work was carried out, the blocks being lowered to the beach by means of derricks.

The design of the groynes is known as the "Owens-Case" type, and consists of a series of columns of 16-in. and 14-in. section and 8 ft. in height, these columns being

View of Sea-Wall and Groyne at Rottingdean.

REINFORCED CONCRETE COAST DEFENCE WORKS IN SUSSEX.

spaced about 5 ft. apart and the intervals filled in with slabs. The piles, founded on a chalk bed, are reinforced with four steel rods, these ranging in diameter from $1\frac{1}{4}$ in. to $\frac{3}{4}$ in. The slabs are similarly reinforced. In the early contracts the columns were strutted on the east side by struts having their lower ends fixed in concrete in a similar way to the columns, but in the latest groynes the struts are omitted and the slabs between columns are made equal to the thickness of the column so as to leave no corners for the shingle to rub against.



Section of Reinforced Concrete Sea-Wall.

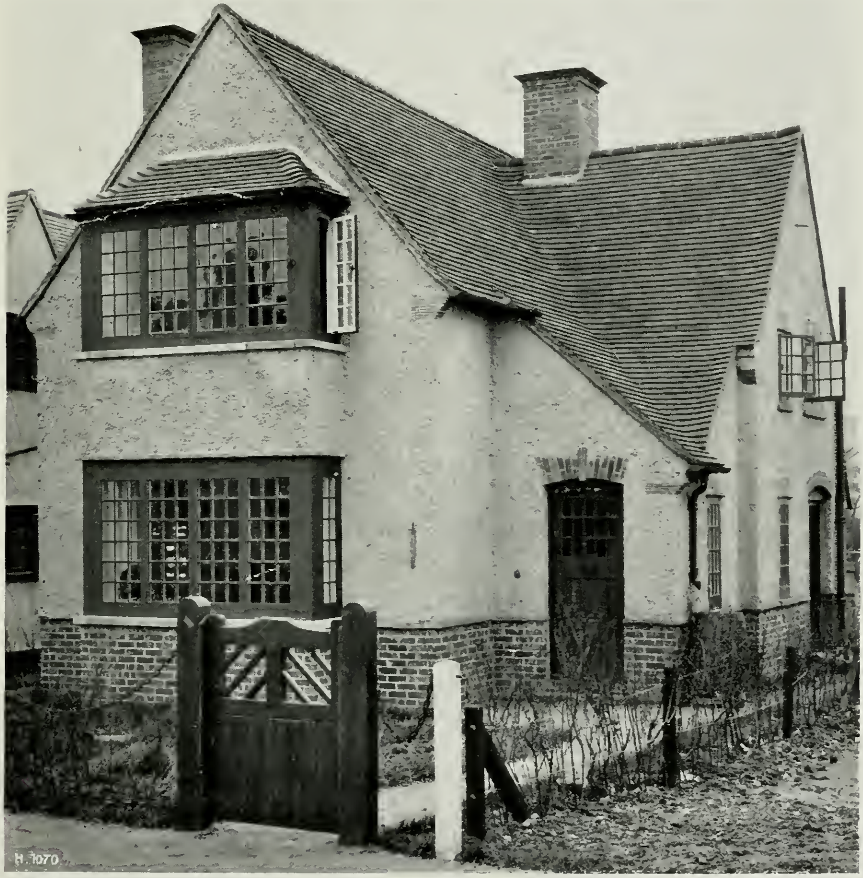
REINFORCED CONCRETE COAST DEFENCE WORKS IN SUSSEX.

A typical section of the sea-walls is shown in the drawing on page 470, each wall being faced for a height of 4 ft. above shore-level with granite concrete blocks tied into the mass concrete work by the reinforcement, the reinforcement extending to the bottom of the foundation concrete.

The reinforcement throughout is on the Hennebique system, and Messrs. Owens and Case, of Westminster, acted in the first instance as Consulting Engineers to the East Sussex County Council, but for the later work Dr. Owens acted alone, the firm having been dissolved. The entire work was done by Messrs. Holloway Brothers (London), Ltd.

ARTISTIC CONCRETE BLOCK HOUSES ON THE GIDEA PARK ESTATE.

On this page will be seen an illustration of one of the houses which are being erected by Messrs. Henry Lovatt, Ltd., of London and Wolverhampton, on the Gidea Park Estate. These houses are erected with "Winget" concrete slabs; they are rough cast on the outside and plastered on the inside. The whole house, with the exception of the chimneys, is built of "Winget" blocks and slabs, and the artistic appearance which can be obtained by this method of construction is plainly seen by the accompanying illustration.

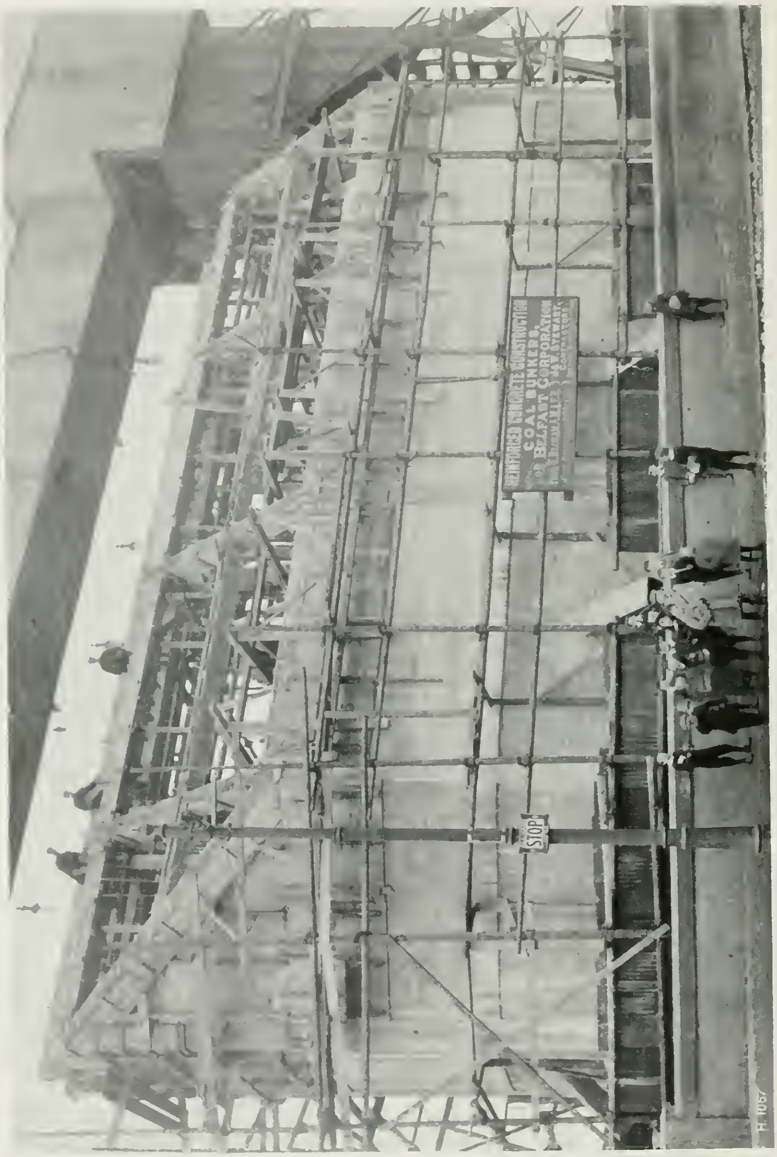


Concrete Block House on Gidea Park Estate.

REINFORCED CONCRETE COAL BUNKERS FOR BELFAST ELECTRICITY WORKS.

The bunkers were built to provide storage for about 2,000 tons of coal, and in such a way as to utilise an already existing overhead bucket conveyor, leading from the quay to the electricity works' boiler-house. The bunkers are divided into four compartments, so that in the event of any overheating taking place it will only be necessary to remove the coal from one compartment. An illustration of the completed structure will be seen in the frontispiece to this issue.

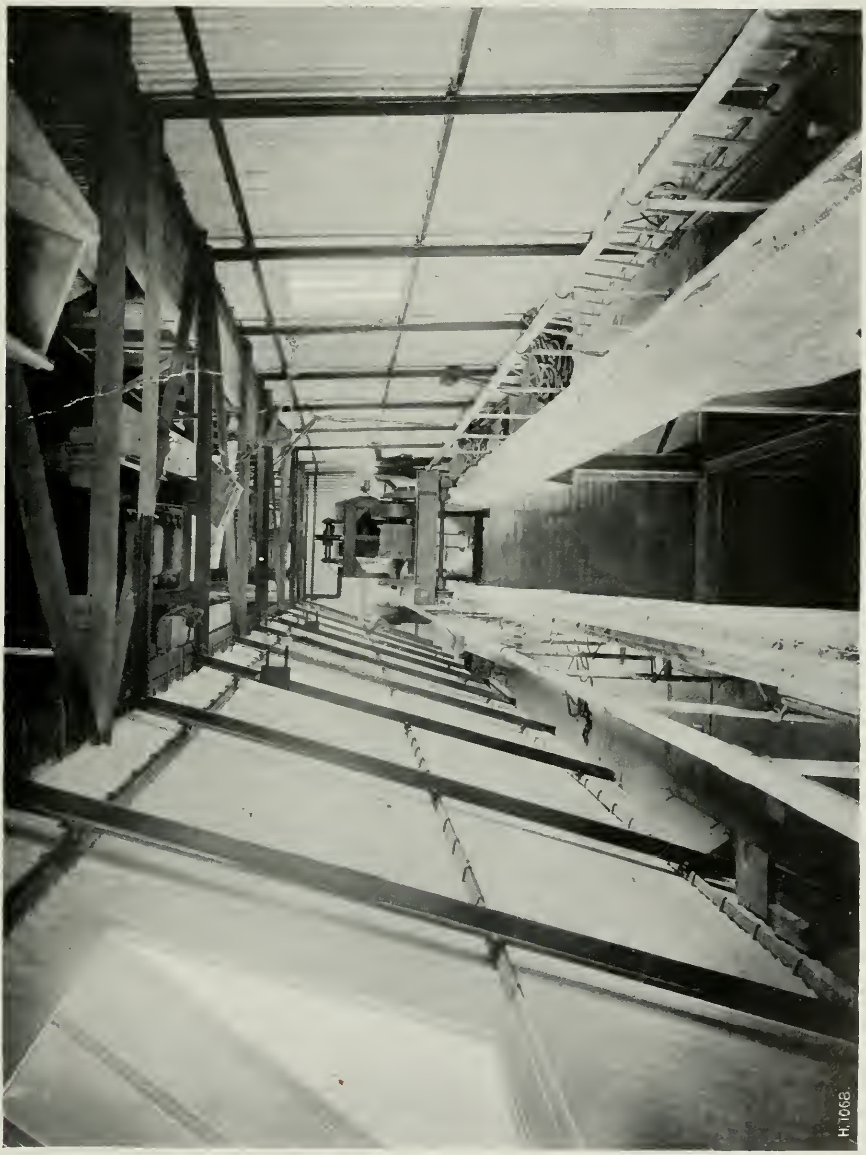
The coal can be taken out from the lower part of the bunkers by means of steel chutes or by steel doors, which are provided at the level of the roadway, but under ordinary circumstances the coal is taken to the boiler-house by means of an electrically-operated travelling hoist (see illustration on next page) in connection with the already existing over-head bucket conveyor.



View of work in progresss.
Reinforced Concrete Coal Bunkers, Belfast.

All the exterior and division walls, varying in thickness from 4 to 6 in., are reinforced with round steel bars, running both horizontally and vertically and forming nettings with square meshes, and the division walls have a system of such netting near either surface of the wall to enable them to withstand the pressure of coal from either side, as often one compartment is full while the next is empty.

The inclined roof slab is only 3 in. thick, and is carried by reinforced concrete purlins and rafters, and the walls are also supported by a system of horizontal beams.



Interior View.
Reinforced Concrete Coal Bunkers, Belfast

The engineer for the work was T. W. Bloxam, A.M.I.C.E., Engineer to the Belfast Corporation Electricity Works, and the work was designed and carried out by Messrs. J. and W. Stewart, of London, Belfast and Dublin.

CORRESPONDENCE.

Under this heading we invite correspondence.

Reinforced Concrete Columns.

May 16th, 1912.

Dear Sir,—Adverting to Mr. Marsh's letter in your May issue. "Interpolation" for intermediate values in the formula:

$$s = 48 - 80\frac{p}{d}$$

is all right, but it does not alter the fact that, for any advantage through the value of "S," in the formula

$$c = c_p[1 - f.s.r]$$

"p" cannot be taken less than ·2*d*.

The advantage in regard to an increase of "r" is not necessarily dependent on the pitch of the laterals. It can be obtained by increasing the diameter of the wires.

Will Mr. Marsh say how the formula for "S" was arrived at?

Yours faithfully,

Harry Jackson.

The Editor, "Concrete and Constructional Engineering."

Mr. Marsh's Reply to Above.

May 20th, 1912.

Dear Sir,—I have your letter of the 18th inst. enclosing copy of Mr. Jackson's letter.

With respect to the value of "r" in the formula

$$c = c_p[1 - f.s.r]$$

it is, of course, obvious that this value can be increased either by using reinforcements of greater area or by closer spacing.

With regard to the derivation of the formula

$$s = 48 - 80\frac{p}{d},$$

the treatment of compression members was discussed by the Committee at very great length, but, speaking shortly, the equation was based on a great number of tests and also on the proceedings of other similar committees. The Committee, in their final decision, were very careful not to go outside the limits of the best modern practice until further tests should have been carried out which would justify them in such procedure.

Yours faithfully,

Charles F. Marsh.

The Editor, "Concrete and Constructional Engineering."

Waterproofing of Concrete.

Aberfan, Glam., May, 1912.

Dear Sir,—Many finely defined and scientific rules have been laid down for the waterproofing of concrete. Some of these are certainly entitled to merit, so far as they go, but are generally beyond the ordinary scope of practice. They are often too expensive, or they require such special supervision and proportioning, that they become impracticable. What is really needed is something handy and practicable in the ordinary sense.

I do not think it possible to make concrete absolutely waterproof, but it can be made waterproof to a degree quite sufficient for all usual requirements, as I have found from many practical experiments I have carried out on different works.

The method is this: Firstly, get the concrete mixed most densely by carefully measuring the voids in the different aggregates and sand, then carefully grading and proportioning same, adding sufficient very finely ground cement just to fill the voids in the sand, with the addition of about 13 per cent. After this is all mixed dry in the ordinary way a suitable clay should be obtained, and mixed in the water for gauging, until the water becomes fairly cloudy. Care should be taken that no small lumps of clay are left in, and care should also be taken in selecting the clay, as some have deleterious properties when mixed with Portland cement. I have usually found a sandy clay best, also china clay, and most clays from Cornwall. To this can be added about 2 per cent. of the weight of cement of well-slaked lime or lime putty, also thoroughly mixed in the water.

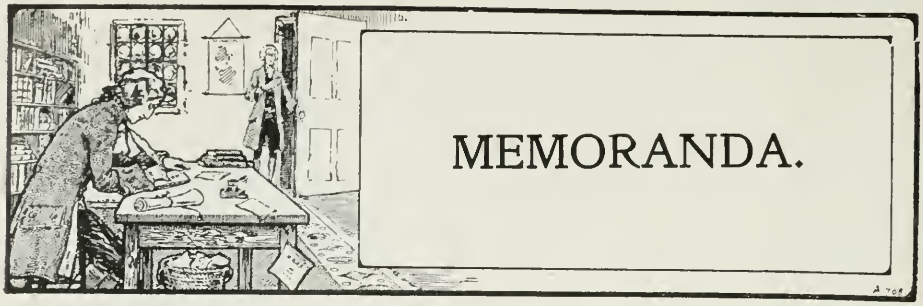
The above method, besides making almost watertight concrete, tends to make a much stronger concrete—at least, so far as my experiments have been concerned, they have all made stronger concretes.

The other remedy of asphalting concrete cannot be called waterproofing, but simply protecting concrete from water, and its efficiency depends on a great many points, and needs special labour to carry out same. The foregoing remarks can be safely relied upon, as any highly scientific process could hope to be carried out in practice, and will not effect the adhesive quality of steel in reinforced work.

Yours faithfully,

George Coe.

The Editor, "Concrete and Constructional Engineering."



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The British Engineers' Association.—This is the title of an Association which was incorporated on April 26th of this year. The organisation of this Association was undertaken by twenty-two of the best-known engineering firms in Great Britain, and its aims are not to serve the interests of any individual firm or group of firms, or to carry on trade in any way, but to establish itself as a national organisation for the promotion on broad lines of the overseas interests of the British engineering industry. Its intention is to employ the great collective influence vested in the engineering industry to encourage the British Government, financial organisations, shipping companies, merchants, and others to promote the interests of British engineering throughout the world in the same manner as the interests of our foreign competitors are promoted by their governments and other bodies.

The efforts of the Association will, in the first place, be directed to China. In that country, especially in view of the new régime, the organisation and strengthening of British engineering interests is urgently needed. The technical education of Chinese engineers on British lines and in the English language will be encouraged.

Every manufacturing engineer in the country, provided that he has no foreign interests which are likely to be detrimental to the objects of the Association, will now be offered an opportunity of becoming a member.

This Association clashes with no other engineering institution or association. It is the first instance of British engineering firms organising themselves on purely national lines to overcome the unfair obstacles which are hampering the trade generally.

The offices of the Association are at 38 and 39, Queen Anne's Chambers, Westminster, S.W., and full particulars will be sent on application to the Secretary at the above address.

A Novel Concrete Reservoir. — At the State Hospital, Yankton, S.D., says *The Times*, a covered concrete reservoir presents the unusual feature of having its roof slab made to serve as the floor of a two-story concrete building. The tank is a rectangle measuring 185 ft. by 45 ft. and is 11 ft. deep. The structure is divided by lines of transverse columns into bays having transverse spans of 15 ft. and longitudinal ones of half that length. The walls were cast solid in 30 ft. sections, the alternate ones being poured. Water-tight joints were secured in these walls by casting a copper strip 12 ft. long and 9 in. wide in the wall sections in such a position that there was an equal bond in the adjacent sections. A slight crimp was made in this copper to prevent it from being stressed by the contraction of the concrete. The floor was cast in slabs 7 ft. 6 in. square, and 1 in. by 4 in. white pine strips painted with asphalte after thorough drying were placed between them. It is expected that these strips will swell and will thus form a water-tight joint. The asphalte coating was applied to prevent the wood from swelling too rapidly.

Pure Cement Concrete as a Lining for Kilns. — Mr. Valeur has found that pure cement concrete possesses many advantages over other materials which have been used as linings for cement kilns. It is only called upon to resist the mechanical wear and tear of the sintered cement mass, which exerts no chemical effect upon it. The concrete is composed of two to three parts by weight of hard, burnt cement clinker (the size of grain being from 2 mm. to 2 mm.=0·08 to 0·8 in.) and one or two parts by weight of ground, slow-setting cement, and is mixed with just enough water to cause the mass to cohere when pressed together. When such freshly-prepared concrete is heated to a temperature corresponding to a red or white heat it hardens quickly, remains hard at high temperatures, and retains its form and hardness, whether it be heated slowly or rapidly, in the sintering zone of a Dietzsch kiln in full work. The concrete begins to set in about an hour, and only about as much as can be used in that time is prepared at one time.

By using an iron ring about 40 cm. = 16 in. smaller diameter than that of the kiln as a mould, a concrete lining 20 cm. = 8 in. thick can be conveniently rammed into the whole length of a rotary kiln. Such a rim of concrete 2·5 metres = 10 ft. wide can be formed in a day and left to set overnight; the next morning the moulding ring is moved and a second concrete rim rammed against the first. Experience shows that no special care is needed in heating the kiln to its full working temperature and condition. A layer of cement concrete only 10 cm. = 4 in. thick which was tried in the sintering zone of a rotary kiln for a period of fourteen days and then cooled, showed that a thin layer of cement had formed on the concrete and split away again on cooling, and the mechanical wear and tear amounted to 3 cm. = 1·2 in.; otherwise the concrete was intact.

As the molten basic cement does not act chemically on the concrete, it does not adhere to the latter as it does to a fireclay lining, except as to the cohesive effect of the ash contained in the coal used for firing.

A cement-concrete lining will last from three to six months in the sintering zone of a kiln, and is also suitable for the end shoot, which can be relined with concrete in an hour without interrupting the work, whereas relining with fireclay necessitates stopping and cooling the kiln. The concrete answers well not only with rotary kilns, but also with other types, such as the Dietzsch and Schneider. The lining can also be composed of blocks of concrete, kept ready for use, and mortared in with cement. The cost of repairing the concrete lining is much less than that of a fireclay, being only about 3s. 6d. to 4s. 2d. per foot, against 68s. 3d. for fireclay lining, and the former is more durable. In actual practice the cost of repairing by the two methods compared as follows, per barrel of cement produced weighing 374 lb. :—

Lining employed in Kiln.	Dietzsch Kiln.	Schneider Kiln.
Fireclay Lining	0·60	0·3
Cement Concrete Lining...	0·08	00·9
Saving per ton of Cement = 6 barrels, in pence ...	3·12	1·26

The process has been adopted in twenty-six works, and is patented in Germany and other countries.—*The Builder.*

An Unusual Combination of Reinforced Concrete and Concrete Blocks.—The E. E. Woodward Building at Rockford, Ill., U.S.A., is particularly notable in that the construction is a combination of reinforced concrete and concrete blocks. The entire frame—including columns, floors and stairways—is of reinforced concrete, while the walls and partitions are constructed of concrete blocks. No wood is used except for the window-frames. It is interesting to note that the blocks carry none of the load of the building and contents except their own weight. The building is 143 by 70 ft., five stories high with basement. The concrete used was proportioned 1 : 2—1/2 : 4, and was entirely mixed by a Ransome mixer. The Simpson Construction Co., of Chicago, were the contractors for the concrete work.

Responsibility for Concrete Mixing.—Mr. A. W. Ransome, of the United States, the well-known manufacturer of mixers, recently stated that he believes that concrete machinery as a class is subjected to more abuse than falls to the lot of any other kind of machinery. In his opinion the hard uses incidental to the service the machines are called upon to perform are often aggravated by neglect. "Make one man responsible for the mixer," was Mr. Ransome's advice, "and allow him one half-hour extra time a day to clean up the mixer and go over carefully the various parts. Where a nut is started, let him tighten it; where a key is loose, drive it home; and, above all, keep an eye constantly on all journals."

Design of Forms for Concrete Work.—It is generally recognised that one of the prime reasons for the comparatively high first cost of reinforced concrete buildings is the cost of the material and labour put in the forms. Therefore continued efforts to

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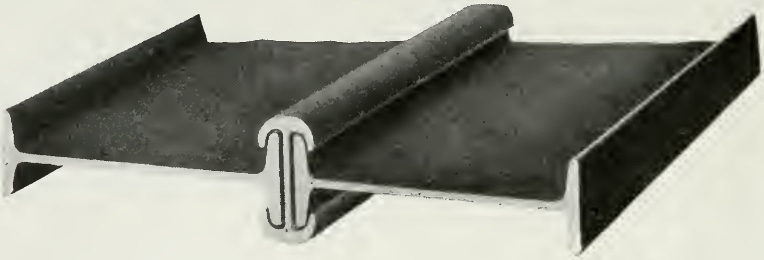

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reduce this item to a minimum are being made by many construction companies. A recent case of the Aberthaw Construction Company, of Boston, is illustrative of the possibilities of saving, even on small jobs, by designing the forms in the drafting room, instead of putting the fitting up in the carpenter's shop. The job in question was a small structure, costing about £5,200, and 50 ft. × 50 ft., four stories high. The design was of the beam and girder type, and was made as regular as possible in order to keep the cost down. In spite of this 113 different kinds of forms were required, making a total of about 600 forms in all, taking about 10,000 sq. ft. of stock. When it is considered that the figures do not include any of the ledges, studs, or anything but the beam and column sides, it will be appreciated that 113 different kinds of forms is a large number for a plain, small job. According to the Aberthaw Construction Co.'s former methods, the carpenters not only made the beam sides but fitted them as they went along, and each one had to be sawed out. A large saving is therefore made by designing the forms for this small job in the drafting room, as the cost of drafting is very much less than the cost of making up the beams and fitting. It is very probable that the failure of new concrete contracting companies is often due to their ignorance of economical methods of handling form work, and the case above is an excellent illustration of how they might go astray on their estimate by not realising the high cost of the many forms on such a small, plain job.

The Advantages of Concrete in the Storage of Petrol.—In a very useful article in a recent number of *The Motor*, dealing with the storage of petrol and some suggestions for its safe handling in the motor house, the writer says: " We now come to the second part of the problem—the storage of petrol. One of the most practical suggestions the writer has seen is that, in a spare corner of the motor house, or outside it, a series of boxes should be built up out of concrete, reinforced with wire netting. These boxes would form a two- or three-tier cupboard, with roof, sides, shelves, and flooring of solid concrete. Iron doors of the self-locking, spring-controlled type, padded with asbestos, should be fitted. From each compartment iron pipes should be run to the top of the building, and their upper ends turned over in syphon fashion. These pipes are for ventilating the store house."

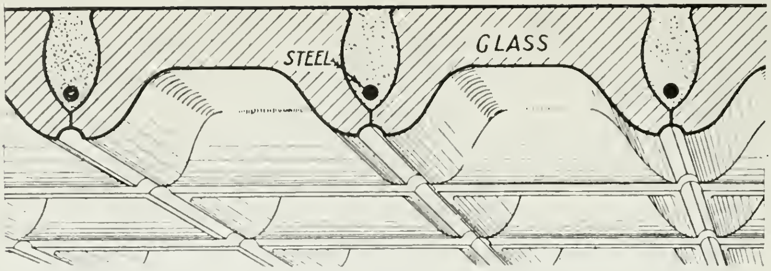
Golf Ball of Cement.—One of the most unique exhibits at the Chicago Cement Show was that of a cement golf ball. The new ball was tried by a former American amateur champion. It is the same size as the regulation golf ball, but with a core of a composition of cork and concrete mixture, which is declared to act in the same manner as rubber.

Unique Use of Concrete.—An unusual use for concrete has been found, according to *The Concrete Age*. This consists of filling the bumper on an automobile with concrete. This device is similar to the usual form of brass tube bumper which extends in front of the fore wheels, lamp, and radiator on the car, and prevents serious injury in the event of a smash-up. The concrete-filled tube is much lighter than a solid bar and considerably stronger than a tubular piece of metal. The extreme hardness of concrete as applied to this device (hydraulically) enables it to stand a much greater blow without breaking than if metal alone were used. Whereas metal will crack and splinter with the impact of a sudden blow, the concrete embedded in its brass casing will not, for in such a form of construction few people realise what amazing strength concrete possesses.

TRADE NOTICES.

"Ferro-Glass" Construction.—This is a new method of floor, roof, and pavement light construction, in which the bars are formed of steel reinforcing rods surrounded by cement, thus preventing the formation of rust. The under side of the cement ribs is glass and can easily be cleaned, consequently there is no maintenance charge for repeated painting of iron frames. Owing to the special construction of the glass covering, most of the rays of light falling on the plates from below are diverted by total reflection, so that the cement joints remain invisible; and at the same time a large number of rays falling on the plates from without penetrate into the interior of the space to be illuminated, thus lighting same. As the steel parts which carry the structure are surrounded by glass and concrete great protection against fire is given.

The glass prisms and steel reinforcing rods are laid on temporary shuttering at the building and the joints are filled by pouring in the concrete, the preparation of special frames therefore becoming unnecessary.



"Ferro-Glass" Construction.

Roofs composed of these prisms and reinforced concrete have the advantage that they lie in the same plane with the roof and do not form any corners or angles in which snow and rain water can collect.

Full particulars of "Ferro-Glass" construction will be sent on application to the proprietors, Messrs. J. A. King and Co., 181, Queen Victoria Street, London, E.C.

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SUBSCRIPTION RATES (Prepaid.—Post free).

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The **Triennial** Subscription is **35/-** (three years).

Address remittances to Concrete Publications, Ltd.

General Offices:
North British and Mercantile Building,
Waterloo Place, London, S.W.

Telegraphic Address: "Concretius London."
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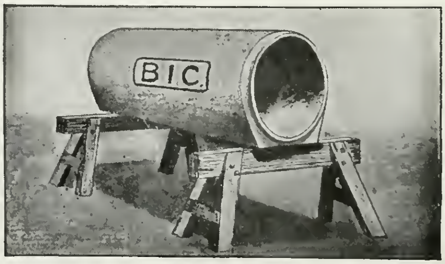
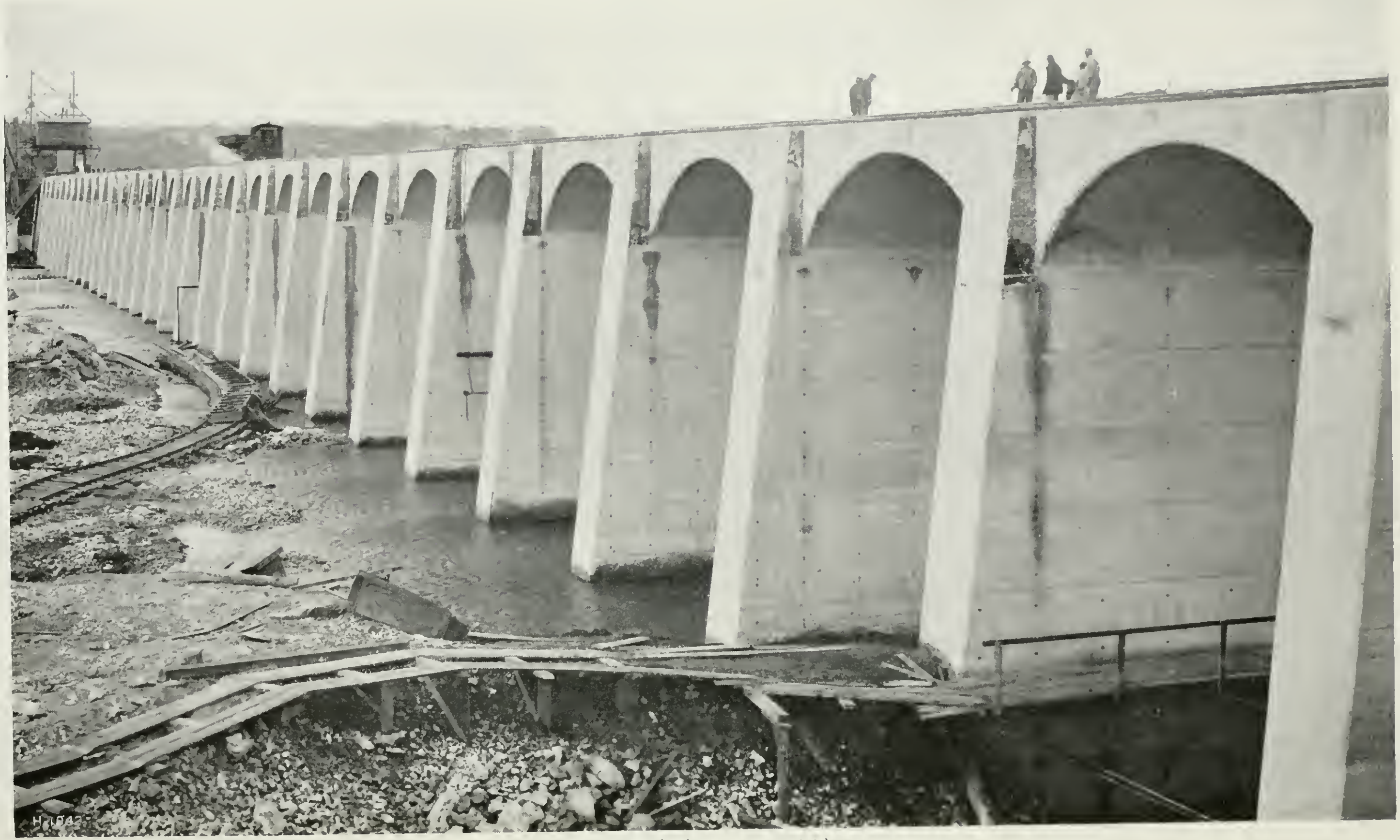


Porous Pipes
Concrete Beams
Hollow Floors

SPECIALITY:—INVENTORS' DESIGNS CARRIED OUT TO ORDER.



(*For description see page* 549.)

Large Concrete Dam for the Mississippi Hydro Electric Plant.

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume VII. No. 7. LONDON, JULY, 1912.

EDITORIAL NOTES.

CEMENT USES.

WHILST we are making but moderate progress in the development of cement uses—and the application of reinforced concrete in particular—the United States of America are making rapid strides in every possible direction where Portland cement can be practically and economically utilised for building and engineering purposes. The enthusiasm, energy and businesslike acumen with which the majority of the members of the architectural and engineering professions of America are adapting modern methods and modern forms of construction for structural purposes—and thereby using Portland cement in largely increasing quantities—is only to be accounted for by the general appreciation of the advantages of Portland cement by the American public, both by residents in the great industrial centres and in the agricultural districts.

A recent echo of this progressive movement in the application of cement, concrete, and of reinforced concrete, may be found in all the technical societies of the American continent; and whether we turn to the proceedings of the National Association of Cement Users of the United States—whose primary purpose is obviously to consider problems relating to the application of Portland cement—or to the conferences of the National Fire Protection Association, the Association of Water Works Engineers, or gatherings such as that of the International Congress of Navigation recently held in Philadelphia, everywhere we find a prominent position accorded to the subject we have so much at heart.

At home, on the other hand, these questions are only dealt with in a very half-hearted sort of manner, and it is as much as the Concrete Institute can do to obtain papers and discussions of practical utility and vitality, whilst enthusiasm is conspicuously absent. We come across occasional papers and discussions taking place in other British societies, but they are presented in a somewhat diffident manner, as if the authorities would wish to excuse themselves for dealing with the subject, and as if the presiding officer of the society were rather ashamed that it had allowed such a subject to be dealt with within its portals.

Now all this diffidence should cease. Reinforced concrete is already a factor of very great importance in the development of the structural works of these Islands and our Colonies. Prejudice against its introduction must cease, for reinforced concrete is not an untried innovation, and prejudice is entirely misplaced, both from an economical and practical point of view. No matter to what extent the country may be asleep in many matters of the utmost importance to its prosperity, those who are engaged in building and structural matters should promptly awaken to the problems of the day, and one of the primary problems amongst these is certainly the application of reinforced concrete for building purposes.

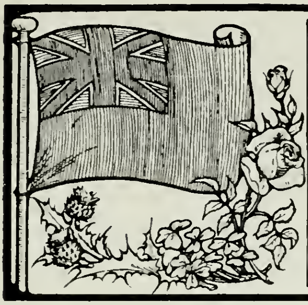
We, on our part, have for several years been diligently analysing the pros and the cons of the uses of concrete and reinforced concrete, but, as has been the case of those best informed on the continent of Europe and in the United States, we have come to the conclusion that the pros greatly outweigh the cons, and that the time has arrived when the energetic development of the uses of these practical materials must be strongly urged.

It is thus to be hoped that readers of this journal will realise that concrete, reinforced concrete, and Portland cement for general use have come to stay, and that the diffidence with which these subjects have been approached when this journal was started is no longer in place. As before, we are open for the publicity of argument against the uses of the material in question, and we shall be careful to advocate such regulations as will limit the possibility of accident and failure. Taken generally, these columns may be held to invite plainly and unmistakably a more thorough, a more energetic, and we hope a more enthusiastic, consideration of the building methods we deal with and which can only be of benefit to this country and its dependencies.

WORK OF AMERICAN TECHNICAL SOCIETIES.

In the preceding commentary we referred to the extraordinary amount of useful information on the subject of reinforced concrete being disseminated in the United States. It might be said that one is entitled to look for such information from the various technical societies, but it is somewhat remarkable to find so much space accorded to the subject at the International Congress of Navigation held in Philadelphia. Obviously local interests in the subject were paramount, but it should also be remembered that a very large proportion of the papers presented at that Congress were put forward by authorities from Europe, and notably from the continent of Europe. The proceedings contained numerous highly interesting reports on the application of reinforced concrete to hydraulic work, a summary of which we are publishing in our current issue. Some exceedingly useful reports were also issued on the most recent works constructed at the more important seaports, with special reference to applications of reinforced concrete therein, and these we shall also deal with at a later date. We would thus strongly recommend those who have not yet joined the International Congress of Navigation to do so now, if only with the object of obtaining the valuable *Proceedings* with which they are concerned.

The important work done by the National Association of Cement Users is well known, and has on several occasions been commented on in this journal. At their recent Annual Convention, however, the papers presented seem to have been of even more technical interest than usual. All the well-known authorities on reinforced concrete contributed to them, and we shall have pleasure in publishing most of these at a later date.



NEW PREMISES FOR MESSRS. WILLIAM WHITELEY, LTD.

By ALBERT LAKEMAN.

Owing to the enormous size of this building, an account of the methods employed in its construction are worthy of attention.—ED.

Some interesting steel construction is to be found in these new premises, which have been erected in Bayswater from the designs of Messrs. Belcher and Joass, and the enormous size of the building lends additional interest. Many difficult constructional problems had to be faced and overcome in order to give the maximum amount of uninterrupted floor space, and also to arrange the steel in such a manner that the fine architectural design was not marred. It is the latter point which usually imposes great restrictions on the engineer, who, naturally, is unable to place the girders and columns to entirely please himself, but must adapt the steelwork to certain levels and positions, and at the same time not exceed certain sizes and depths.

The completed portion has a frontage to Queen's Road of about 320 ft. and a return frontage to Porchester Gardens of about 162 ft., while the general depth of the site is about 130 ft. There is one floor below the pavement at a depth of about 12 ft. and four floors above the pavement level, giving a total height of 78 ft. The only reinforced concrete work is that employed in the retaining walls, the remainder of the building being constructed of steel. There were about 3,4000 tons of the latter material employed and there are over ninety stancheons.

The foundations generally were calculated to give a maximum load of three tons per square foot upon the soil, the latter being a good hard clay. Grillages were formed under all stancheons carrying large loads, and these were built up with two layers of rolled steel joists enveloped in concrete, the largest grillage being 17 ft. by 14 ft., and consisting of a top layer of six 24-in. by 7½-in. by 100-lb. rolled steel joists, and a bottom layer of fifteen 16-in. by 6-in. by 62-lb. rolled steel joists. Both layers were stiffened by rows of ferrules and bolts to form a good stiff foundation. The smallest type of grillage was 8 ft. square, and this had a top layer of four 14-in. by 6-in. by 57-lb. rolled steel joists and a bottom layer of nine 9-in. by 4-in. by 21-lb. rolled steel joists.

The reinforced concrete retaining wall, which was constructed under the pavement next Queen's Road and Porchester Gardens, is illustrated in *Fig.* 1, and it will be seen that the roof to the vaults is formed in conjunction with the wall to give a monolithic construction. The toe of the wall has a projection of about 12 ft. with a maximum thickness of about 1 ft. 9 in. and a minimum thickness of about 12 in., with reinforcing bars at about 6-in. centres for one half the projection and about 12-in. centres for the remaining half in both upper and lower surfaces, while distribution bars are provided at 2-ft. centres. The vertical wall has a thickness of about 1 ft. 9 in. at the base, diminishing to about 1 ft. 3 in. at the top, with reinforcing bars at 6-in. centres in both inner and outer surfaces and distribution bars similar to those in the toe. The vault roof has a projection from the wall of about 8 ft. 6 in., and this is reinforced in a similar manner to the toe. Additional bars are placed at the angles, as shown in the drawings, which also illustrate the method employed when the toe was adjacent to the column foundation blocks. The height of the wall varied somewhat on account of the fall in the pavement.

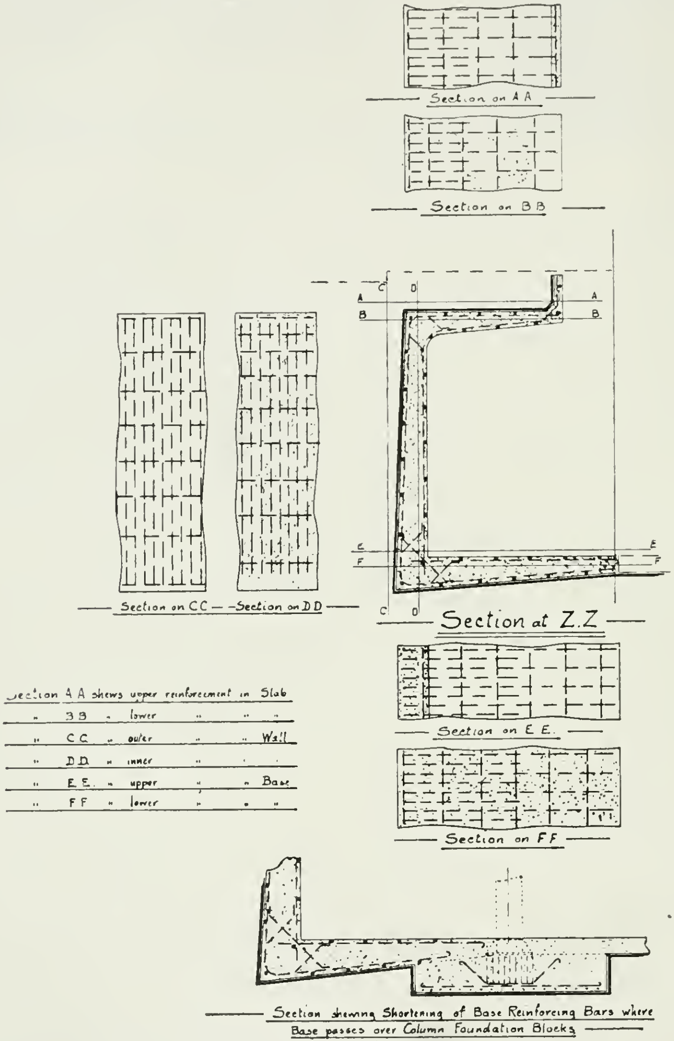
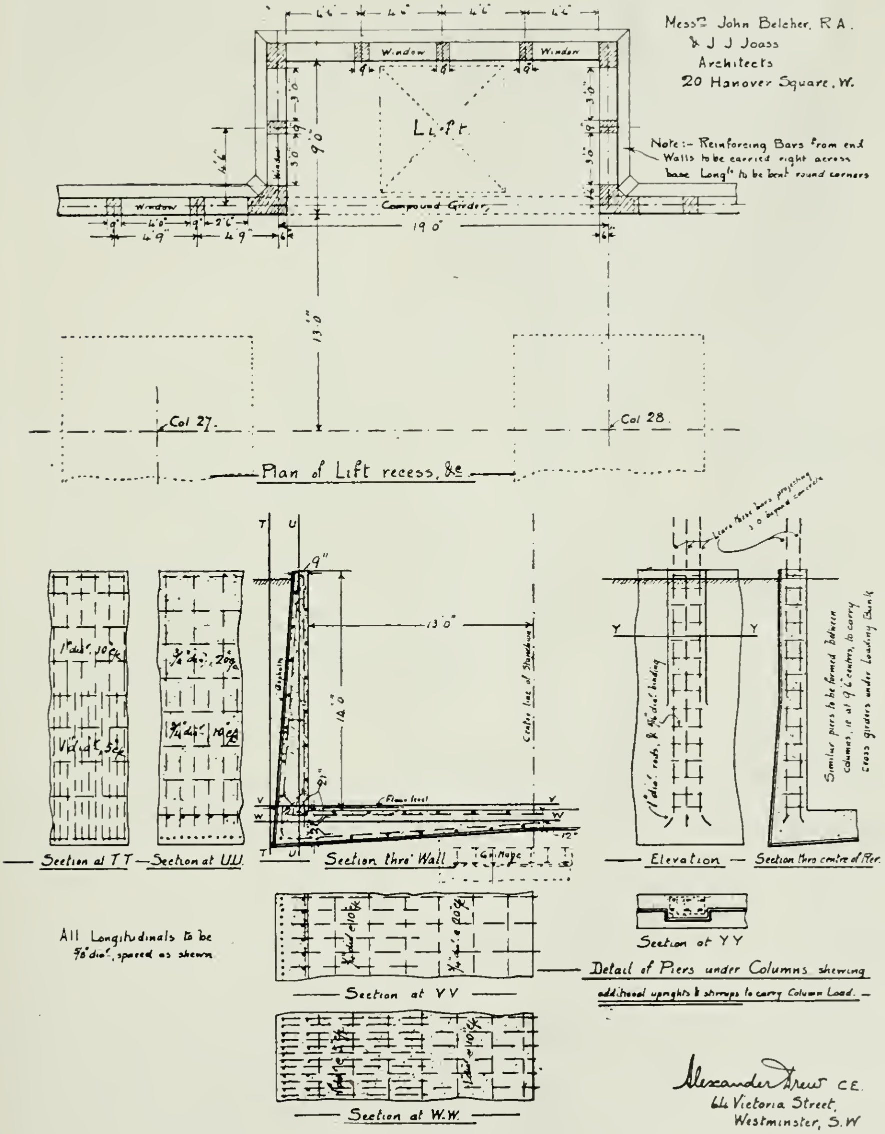


Fig. 1. Reinforced Concrete Retaining Wall next street.
MESSRS. WHITELEY'S NEW PREMISES.

Another instance of a reinforced wall is that illustrated in *Fig.* 2. This was constructed at the rear under a loading bank in the dispatch department. This wall is 14 ft. high, with a maximum thickness of 1 ft. 9 in. and a minimum thickness of 9 in. The outer surface is reinforced with 1-in. diam. bars, spaced at 5-in. centres in the lower part and 10-in. centres in the upper part, while the inner surface is reinforced with $\frac{3}{4}$-in. diam. rods at 10-in. and 20-in. centres.



Messrs John Belcher, R A.
& J J Joass
Architects
20 Hanover Square, W.
Window
Window
Lift
Compound Girder
19 0"
13 0"
Note:– Reinforcing Bars from end Walls to be carried right across base Long'ls to be bent round corners
Col 27
Col 28
Plan of Lift recess, &c
Section at TT
Section at UU
Section thro' Wall
Floor level
Centre line of Stanchions
14 0"
9"
21"
Elevation
Section thro centre of Pier
Similar piers to be formed between columns, ie at 9' 6" centres, to carry cross girders under Loading Bank
All Longitudinals to be 5/8" dia. spaced as shewn
Section at VV
Section of YY
Detail of Piers under Columns shewing additional uprights & stirrups to carry Column Load.
Section at W.W.
Alexander Drew C.E.
64 Victoria Street,
Westminster, S.W

Fig. 2. Retaining Wall at rear.
MESSRS. WHITELEY'S NEW PREMISES.

The toe has a projection of 13 ft. with a minimum thickness of 12 in., and is reinforced in a manner corresponding to that employed for the vertical portion of the wall. The wall was not carried right through in a straight line, but recessed for a lift, as shown.

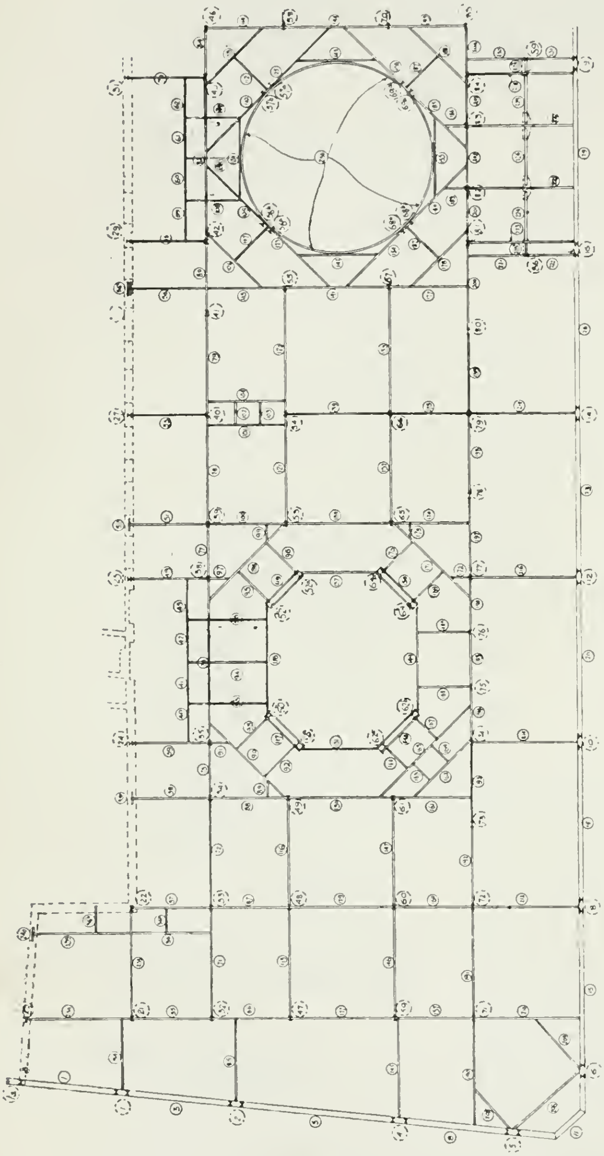


Fig. 3. Plan showing steelwork at second floor level.
Messrs. Whiteley's New Premises.

The general lay-out of the steel is illustrated in *Fig.* 3, which shows the work at the second floor level. It will be seen that the framing in the interior of the building was governed to a large extent by the large open wells, which form a feature of the planning, and are terminated at the roof with large steel and glass domes. The front stancheons are spaced at intervals of about 46 ft., and all the main girders in the façades were placed at the second floor level, this being necessitated by the character of the design. Owing to the large distance between the stancheons and the impossibility of using large girders at the first floor level, two intermediate channel suspenders were placed in each bay, these being carried down from the main girders at the second floor and permitting the use of ordinary rolled steel joists at the first floor. The depth of the girders at the second floor was restricted by the stonework, and large loads had to be carried at this level in addition to the first floor loads brought upon them by the suspenders. The section employed consisted at the centre of the span of two ½-in. web plates, having a distance apart of 11½ in. and a depth of 30 in., with four angles and five flange plates 24 in. wide at top and bottom, while at the ends of the girders the section was reduced to one plate at top and bottom and two ¾-in. web plates. T-stiffeners, 6 in. by 4 in. by ½ in., were provided at 3 ft. 6 in. distances along the web. There were five girders of this type, four being required on the Queen's Road frontage and one on the frontage to Porchester Gardens. The end girders at the intersection of the frontages were varied on account of their being cantilevered for a distance of over 8 ft. to carry the corner of the building, that on the Queen's Road side having a total length of over 55 ft. Some idea of the loads brought upon these girders by the stonework can be gathered from the fact that the cornice at the second floor level weighed one ton per foot run, the top cornice and entablature a similar amount, and the granite columns were about sixteen tons each. The method of carrying the top cornice and entablature is shown in *Fig.* 4, where it will be seen that the tailing down of the cornice is assisted by copper straps about 2 ft. apart attached to one of a pair of rolled steel joists, and the architrave and soffit is hung up with copper T-hooks placed in each joint. The total load on one span was about 35 tons, and it is so situated that of this amount 26¼ tons is carried by the front joist and only 8¾ tons by the back one. The joists were connected by separators, and open joints were left in the stonework where necessary to prevent any possibility of the edges being spalled.

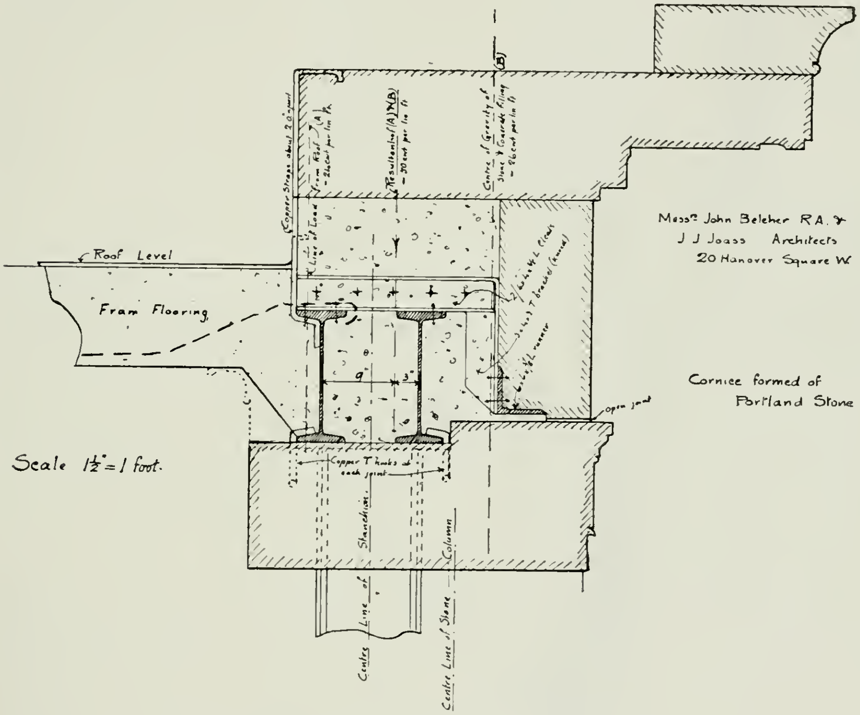

Copper Straps about 2 0" apart
Line of Load from Roof (A) = 26 cwt per lin ft
Resultant of (A) & (B) = 50 cwt per lin ft
Centre of Gravity of Stone & Concrete Filling (B) = 26 cwt per lin ft
Roof Level
Fram Flooring
9"
3"
Copper T hooks at each joint
Centre Line of Stanchion
Centre Line of Stone Column
open joint
Messrs John Belcher R.A. & J J Joass Architects 20 Hanover Square W.
Cornice formed of Portland Stone
Scale 1½" = 1 foot.

Fig. 4. Method of carrying Top Cornice and Entablature.
MESSRS. WHITELEY'S NEW PREMISES.

The largest girder in the whole building is that situated at the second floor level over the main entrance from Queen's Road, and this has a span of about

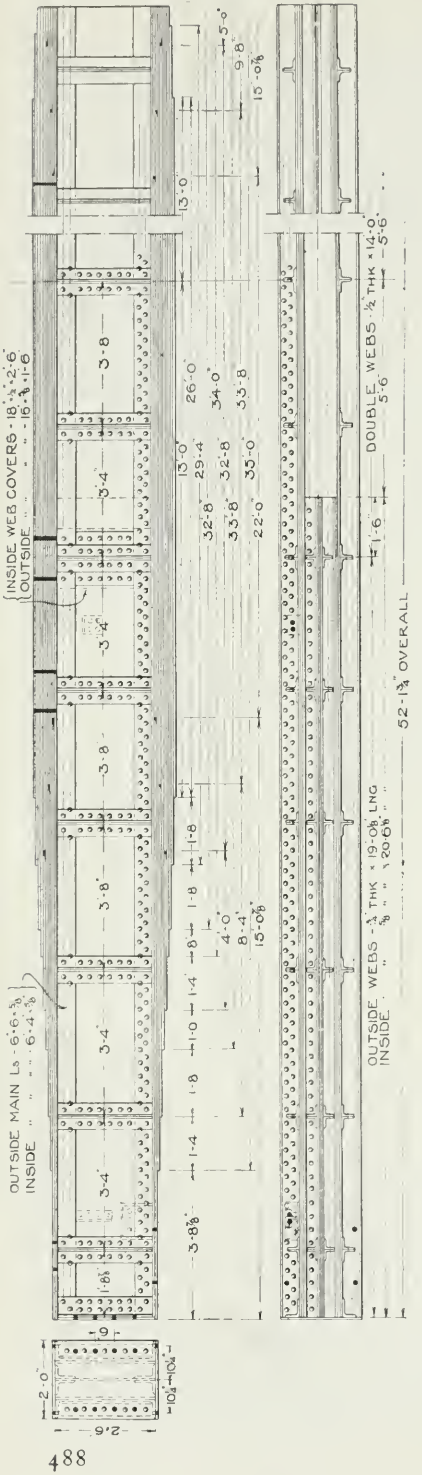

INSIDE WEB COVERS - 18'-½"-2'-6"
OUTSIDE " " - 16'-⅜"-1'-6"
OUTSIDE MAIN Ls - 6'-6-⅝"
INSIDE " " - 6'-4-⅝"
OUTSIDE WEBS - ¾" THK × 19'-0⅛" LNG
INSIDE " ⅝" " × 20'-6⅛"
DOUBLE WEBS ½" THK × 14'-0"
52'-1¾" OVERALL
3'-8
3'-4
2'-0"
2'-6

Fig. 5. Details of Girder.

Messrs. Whiteley's New Premises.

50 ft. and carries two concentrated loads of 85 tons each, two concentrated loads of 90 tons each, and a distributed load of 90 tons, giving a total load of 440 tons, exclusive of the weight of the girder itself, which was 31 tons. The concentrated loads are brought on the girder by two pairs of stancheons carrying the upper part of the building. It is worthy of note that although the girder weighed 31 tons it was raised from the pavement level to its position, a height of 34 ft., in about seven minutes. The section at the centre consists of three web plates $\frac{3}{4}$ in. and $\frac{5}{8}$ in. thick, with eight angles 6 in. by 6 in. by $\frac{5}{8}$ in. and 6 in. by 4 in. by $\frac{5}{8}$ in., and nine flange plates at top and bottom, each being 24 in. wide and $\frac{3}{4}$ in. thick, the depth over the angles being 2 ft. 6 in. Stiffeners were provided on the inside and outside webs at about 3-ft. 8-in. centres, as shown in the drawing of this girder in *Fig.* 5. This is an exceptionally large girder for a building of this class, and its magnitude may be said to represent the scale of the work throughout, which is bold and striking.

The drawings in *Fig.* 6 illustrate the method of framing at the roof level to carry the corner dome, and it will be seen that the girders which carry the extreme outside corners are actually hung up to the fulcrum girder instead of resting upon the upper flange in the usual manner. This method was necessitated by certain fixed levels, and it is ingenious while being practical and safe. The fulcrum beam is composed of two 24-in. by $7\frac{1}{2}$-in. by 100-lb. rolled steel joists, with three 16-in. by $\frac{3}{4}$-in. plates at top and bottom, while the cantilevered beams are made up with one 18-in. by 7-in. by 75-lb. rolled steel joists, and

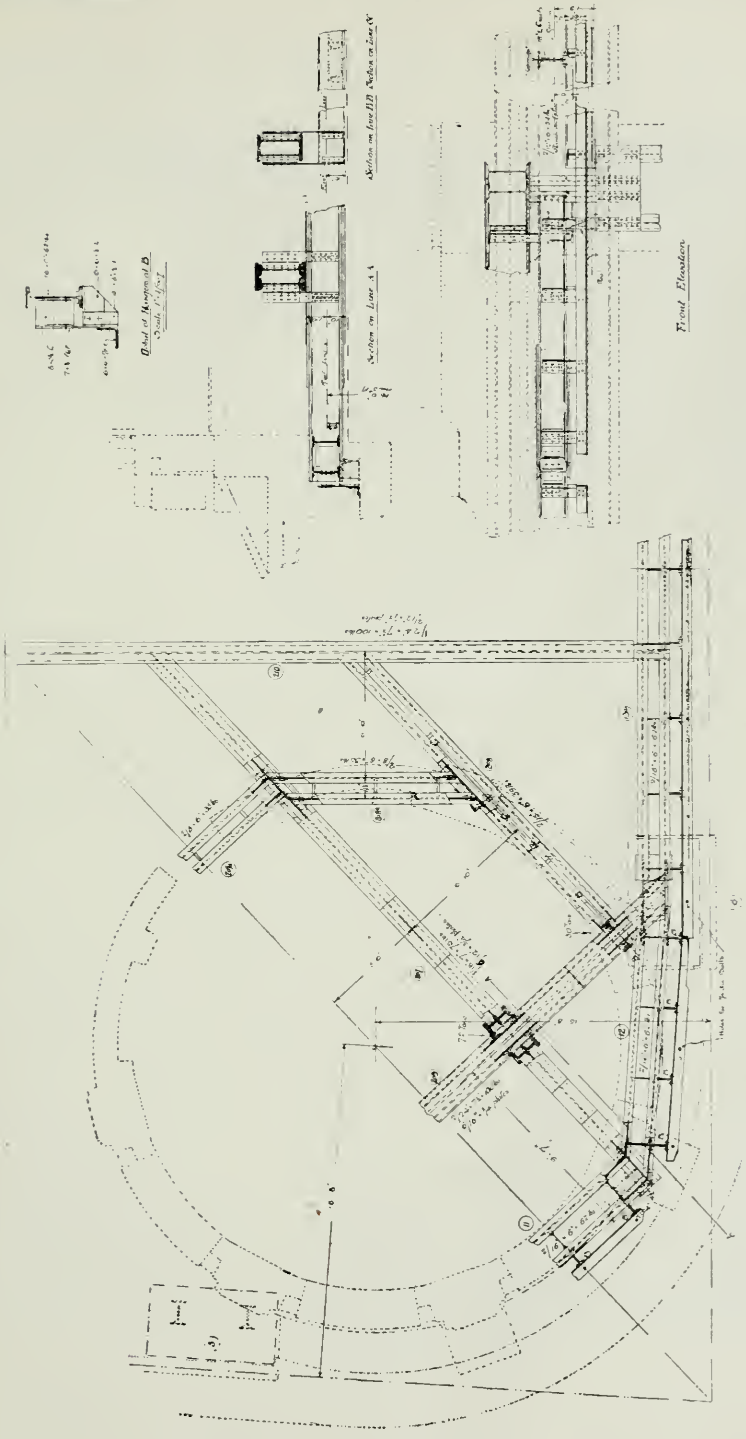


Fig. 6. Steelwork supporting Corner Dome
MESSRS. WHITELEY'S NEW PREMISES.

three 12-in. by $\frac{3}{4}$-in. plates at top and bottom; the ends of these latter are connected by two 16-in. by 6-in. by 62-lb. rolled steel joists, which come immediately

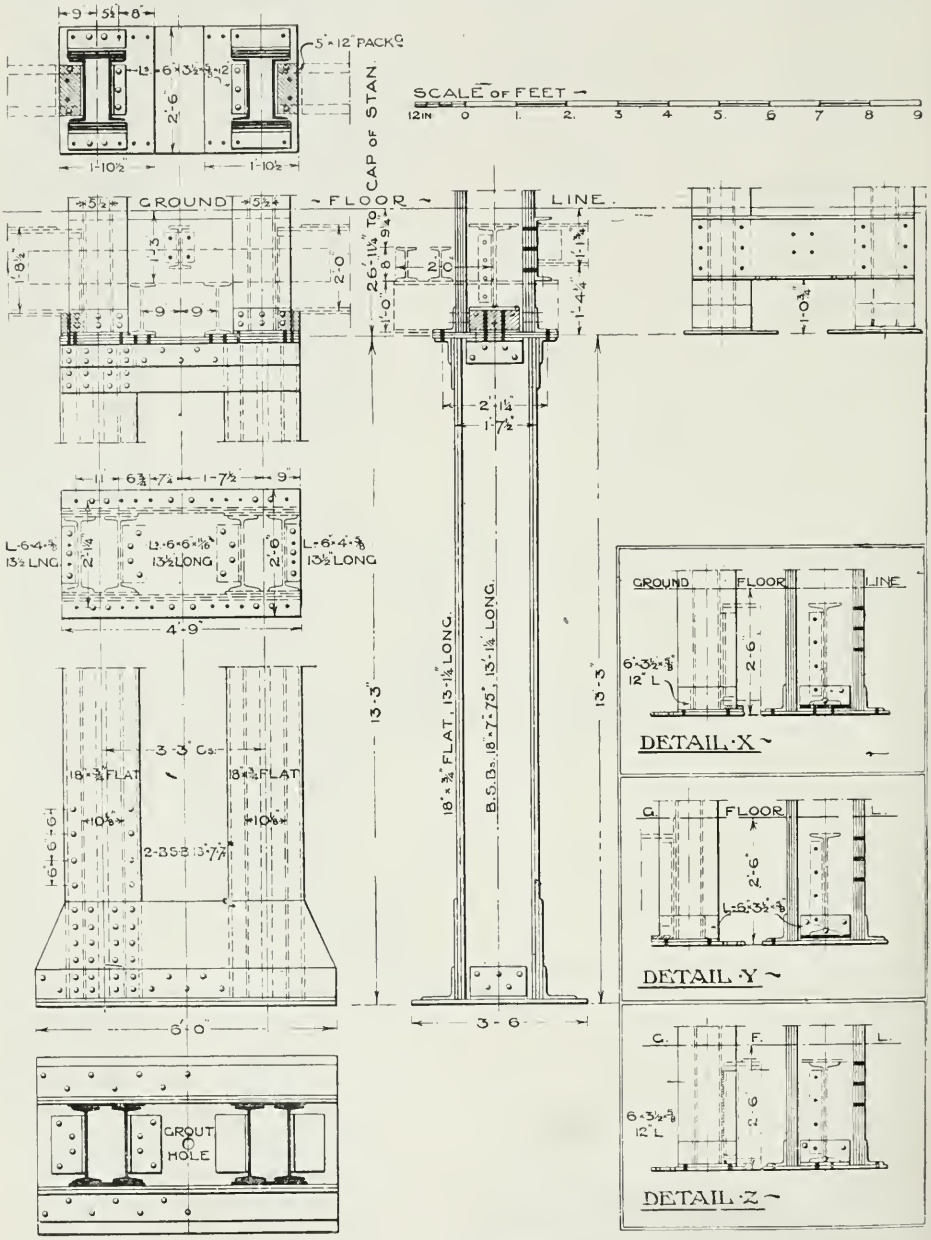


Fig. 7. Details of Stancheons.
MESSRS. WHITELEY'S NEW PREMISES.

under the stone cornice and carry same. The column loads in some instances are very heavy, rising up to as much as 826 tons; the ordinary front stancheons carrying 570 tons each and the twin columns, also situated in the front, having a load of 647 tons.

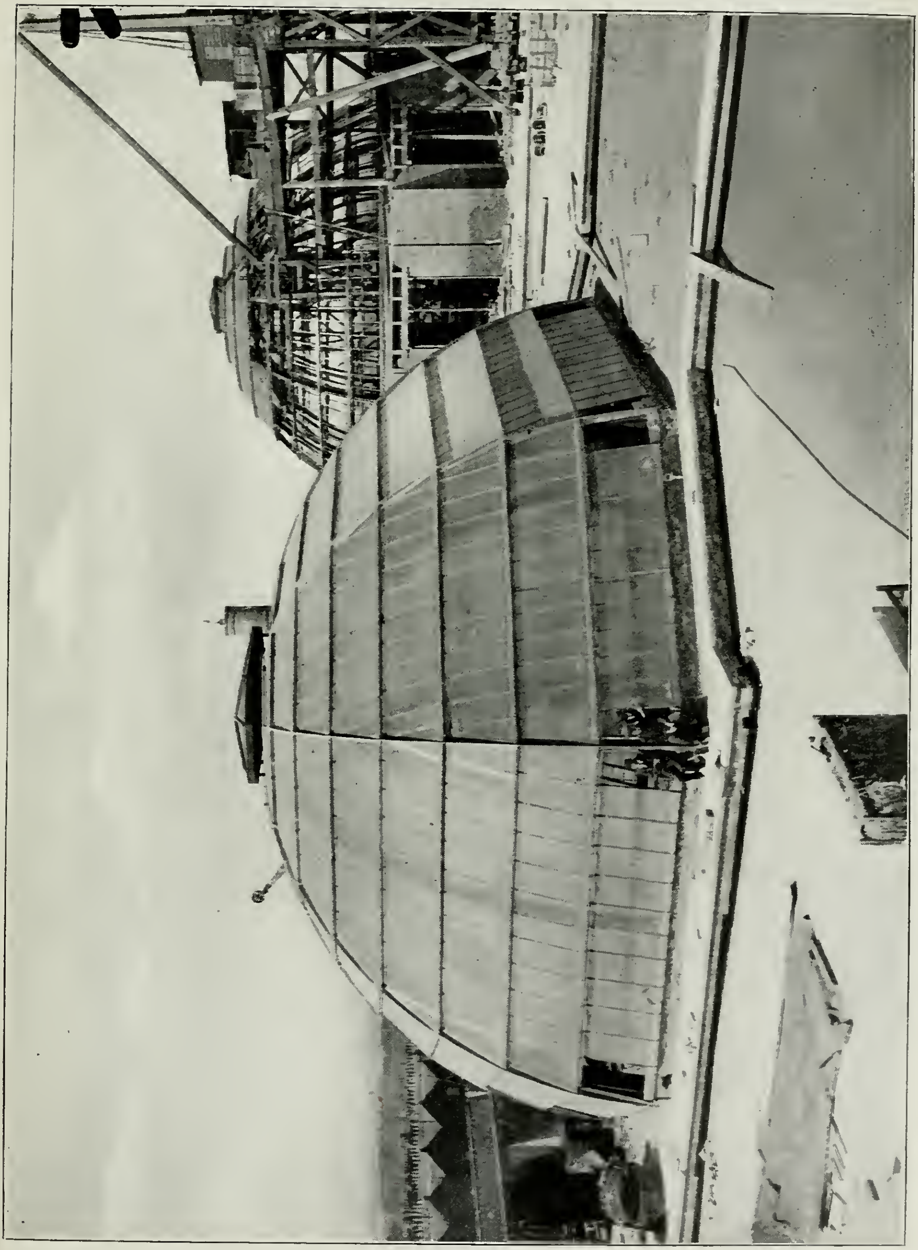


Fig. 8 The Domes in course of construction.
Messrs. Whiteley's New Premises.

The type of stancheon employed in the front walls consisted in many cases of two channels riveted together with flange plates; a typical section consisting of two 15-in. by 4-in. channels, four 14-in. by $\frac{5}{8}$-in. plates, and two 14-in. by 2-in. plates. The drawing illustrated in *Fig.* 7 shows a typical instance of the twin stancheons, which were placed with a distance of 3 ft. 3 in. between the centres with connecting gusset plates at the head and base. Each stancheon is

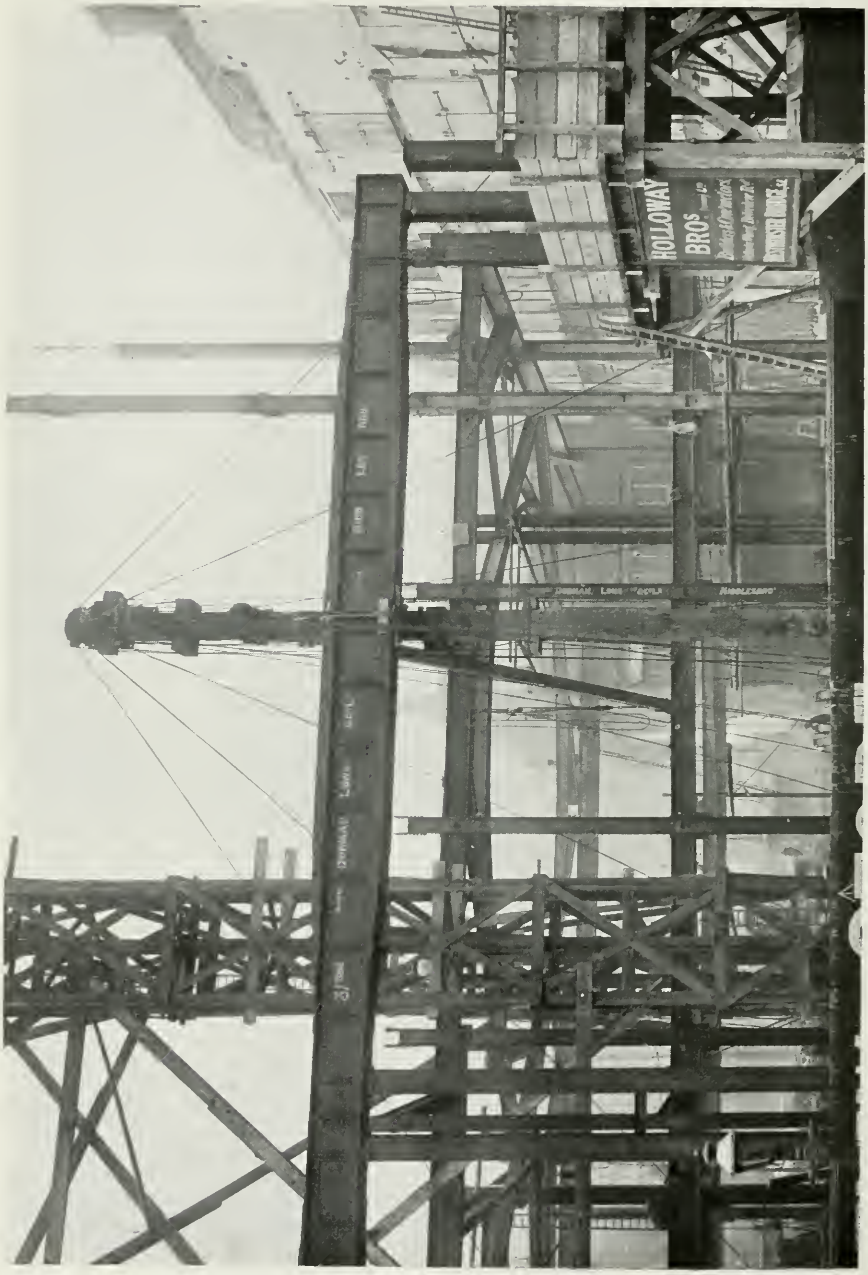


Fig 9. The largest Girder in position.
MESSRS. WHITELEY'S NEW PREMISES.

composed of two rolled steel joists and flange plates below the ground floor level and two steel channels and flange plates above this point. This method gave a stiff and economical form of stancheon, which was also applicable for its position in the stonework, each of the pair being situated behind one of the pair of granite columns, which are a feature of the façade. The compound steel stancheons employed in the interior of the building were built up with three rolled steel joists and flange plates, one steel joist being placed at right angles to the other two and being connected to the webs, giving a stiff section suitable for the heavy loads that had to be carried.

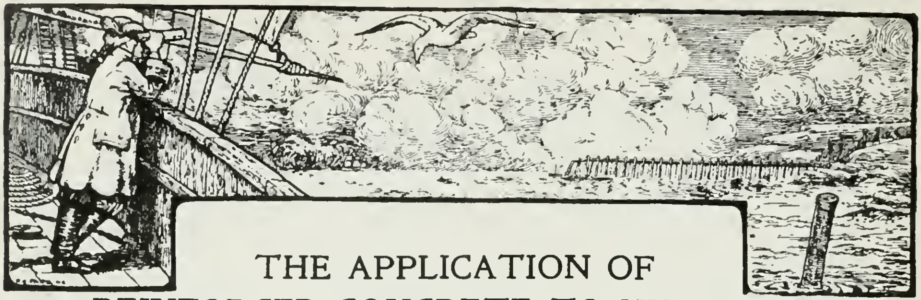
A typical section is that composed of two 20-in. by 7½-in. by 89-lb. rolled steel joists, one 14-in. by 6-in. by 57-lb. rolled steel joist, and four 24-in. by ½-in. flange plates. A stancheon of this size required a base plate 5 ft. by 3 ft. 3 in. and 1¼ in. thick, while the ¾-in. gusset plates connecting the stancheon to the base plate required to be stiffened on the splayed edges by steel angles to prevent any buckling tendency.

The floors were constructed to carry a total load of 2 cwt. per foot super, and executed on the hollow block system of the Fram Fireproof Construction Co. A certain portion of the floor load was carried on the main back wall, but the latter was in turn carried on large girders at the ground floor level and did not go right down to the soil. One of these large girders had a span of 30 ft., with a depth of 2 ft. 6 in. over the angles, the section at the centre of the span consisting of two web plates ⅝ in. thick and five flange plates top and bottom 20¼ in. wide, the thickness being either ⅝ in. or ¾ in. Web stiffeners were provided at 3 ft. centres.

Owing to the large area of the building it had to be divided up into sections by party walls to comply with the L.C.C. requirements, and the whole of the work is fire-resisting in character. There are innumerable items of constructional interest, and it must be considered as an excellent example of a steel framed building, which has been designed in a scientific manner, with due regard to all the restrictions which must necessarily be imposed in a building intended for commercial purposes, and it reflects great credit upon those responsible for its conception and execution.

The glazing for the domes, as well as the whole of the steelwork for domes, was provided and fixed by the British Challenge Glazing Co., Ltd.

The whole of the steel construction was designed by Mr. Alexander Drew, of Victoria Street, who acted as consulting engineer to the architects, and the steelwork contractors were Messrs. Dorman, Long and Co., of Middlesborough, the general contractors being Messrs. Holloway Bros., Ltd., of London.



THE APPLICATION OF REINFORCED CONCRETE TO HYDRAULIC WORKS.

GENERAL REPORT by MR. JOHN STEPHEN SEWELL.
Formerly Major, Corps of Engineers, U.S.A., Gantt's Quarry, Ala.

The XIIth International Congress of Navigation was held in Philadelphia in May last, when a number of useful papers were presented. We give below the report of Mr. J. S. Sewell, summarising the five communications presented on the subject of the application of reinforced concrete to hydraulic work.—ED.

FIVE communications on this subject were submitted to the Congress, as follows:—

No. 31. By M. Jacquinot, Chief Engineer of Ponts et Chaussées, Chaumont (Haute-Marne), France.

No. 32. By Mr. R. W. Vawdrey, A.M.Inst.C.E., Portscatho, Parkhill Road, Sidcup, England.

No. 33. By the Hungarian State Water Survey.

No. 34. By M. Mederico Perilli, Chief Engineer of the Genio Civile, Ravenna, Italy.

No. 30. By Mr. Richard L. Humphrey, President, National Association of Cement Users, United States of America.

A summary of the five papers so far received follows:—

SUMMARY OF PAPER BY M. JACQUINOT.

M. Jacquinot describes, in general terms, a reinforced concrete water tank erected in 1875, which has been in successful use since that date. The favourable bearing of this case in the question of durability under water pressure is evident. He then gives a short summary of the use of reinforced concrete in France. He describes some interesting encorbelments or cantilever constructions, carried out in connection with the enlargement of the St. Lazare railway station in France. While these were not works of navigation, the applicability of such forms to possible cases connected with navigation is evident.

M. Jacquinot also describes some highly interesting applications of dense concrete, not reinforced, for the purpose of cutting off infiltrations through permeable soils, and through dykes of both earth and masonry.

In the first case, a wall of hollow blocks made of very dense, well-rammed concrete was built in a trench dug down through the permeable soils.

The slopes of the earthen dykes were riveted with slabs of similar material, and the upstream faces of the masonry dykes were faced with it. The care and ingenuity expended in making the joints secure against percolating water, and in providing a harmless exit for any water that might leak through, are worthy of special attention; the success of the work, in all cases, seems to be assured. In connection with the facing of the masonry dyke of the Mouche reservoir, two experiments with reinforced concrete are described. While the types experimented with were not finally adopted, there seems no reason why the last one of the two, at any rate, might not have been made successful.

The dykes above referred to all pertained to reservoirs connected with the Marne-Saone Canal. There is also a description of a thin reinforced facing of the same dense concrete, applied to some rubble quay walls connected with the same canal, to render them watertight. A coating of hot tar was applied to the finished work. It is entirely successful.

Reference is also made to a reinforced screening wall, constructed in 1903, and reported to the St. Petersburg Congress, which is still behaving satisfactorily. This wall was built at the Settons dyke. Here, also, hot tar has been found a useful adjunct.

Three interesting dykes, or dams, in the French Pyrénées are described, in all of which reinforced concrete is used for a water-tight revetment, and, in one case, for the construction of a gallery which affords access to the rear face of the foot wall for inspection and repairs; presumably this gallery may be useful also for intercepting and carrying away any percolating water.

In some of the revetments and watertight barriers and facings described by M. Jacquinot a few insignificant cracks have appeared, but apparently they are no greater than would have been inevitable in jointed masonry walls of equal extent. Although some of these works are exposed in a severe climate, no ill-effect of frost at the water line is apparent, which shows that it is possible to make such work secure under conditions that have caused some failures and many misgivings hitherto. The only requisites for success seem to be care and intelligence in design and execution.

M. Jacquinot also describes a very interesting aqueduct, partly closed and partly open, designed to carry, apparently, both storm water and sewage. The foundation was very soft, and full advantage was taken of the structural qualities of reinforced concrete to reduce the weight to a minimum. After several months' use, no deterioration is apparent, but, in this case, it will be interesting to hear, at future meetings of the Congress, whether the sewage has had any deleterious chemical effects on the cement. The invert has been protected from abrasive action by a masonry facing. This is common practice, but it may be asked whether the dense rammed concrete described elsewhere would not have been cheaper and equally effective. A number of other reinforced concrete conduits are described, together with an ingenious form of movable centering used in one case. All of these works have been successful, and as some of them are subjected to heavy pressure, they confirm the favourable conclusions to be drawn from the behaviour of the water tank first mentioned.

M. Jacquinot describes a number of instances of the successful use of reinforced concrete for canal troughs and revetments of embankments, in both irrigation and water power developments. The works are all successful; the cracks and leaks are all insignificant—probably not greater than would be experienced with any type of structure hitherto used for similar purposes. Two successful reinforced concrete dams are briefly described and reference is made to some sills or submerged weirs constructed for regulating the gradients of mountain streams. The latter have suffered from the abrasive action of gravel, and fears are expressed that the same cause may injure some of the works pertaining to Alpine power plants.

One of the most interesting applications of reinforced concrete described by M. Jacquinot consists of the construction of some gangways (towpaths) in the tunnels of the Nord Canal. The material seems to have found here an ideal application in a field belonging to itself alone.

Of equal interest are the brief descriptions of quay walls and jetties of various types, of the reinforced concrete arch built over the St. Martin Canal where it passes through Paris, and of the jetty for the State Railway at Bordeaux. A number of bridges are mentioned, with brief descriptions of interesting features.

The conclusion presented with this communication is an argument for the fearless use of reinforced concrete under the guidance of experience and good judgment, and for the development of its many possibilities in connection with navigation works, not omitting the question of artistic appearance.

MR. VAWDREY'S PAPER.

Mr. Vawdrey states that the application of reinforced concrete to works connected with inland navigation has not been so extensive in Great Britain as in other countries owing to the decline of inland navigation itself, and also to the fact that the materials for concrete in mass are generally so cheap and abundant that the economy of reinforced concrete has not been apparent.

He states, however, that the extent to which it has found application in connection with the Manchester Ship Canal is an indication that it will be freely used in the event of a revival of inland navigation generally.

Bridges — Mr. Vawdrey describes a reinforced concrete bridge at Warrington, over the Mersey, designed and built on the Considère system. It is a structure of considerable magnitude, and is interesting in both design and execution. It had temporary hinges at the springing lines and the crown to permit the bridge to adjust itself to the dead load. These were eliminated by work done subsequent to the striking of the centres.

Two bridges just completed at Blackburn, and a third just begun at Burnley, over the Leeds and Liverpool Canal, are also described. They are reinforced with Indented bars. The arches are made continuous with the abutments, whereby they could be designed of lighter section and afford more headroom. In these bridges steel with a yield point of 50,000 lb. per sq. in. was used, and apparently the working stresses were proportioned to this high figure. Mr. Vawdrey argues that as this steel is possessed of great ductility there can be no objection to its use, as it is not liable to break in bending.

Mr. Vawdrey also briefly refers to several other successful bridges in reinforced concrete.

Piers and Jetties.—One pier is described and illustrated by Mr. Vawdrey. It was built at Gravesend on the Thames for the Amalgamated Press, Ltd. The reinforced concrete piles were driven in pairs, and the outer ones were stiffened against buckling and shocks by enclosing each pair in a cylinder of reinforced concrete.

Warehouses, etc.—Under this head Mr. Vawdrey states that many such structures have been built in maritime ports, but in connection with inland navigation he briefly describes only the warehouses connected with the Manchester Ship Canal, having 740,000 sq. ft. of floor area, and designed for heavy loads. They are built on the Hennebique system.

Water Supply.—As most of the canal reservoirs constructed in England were built some time ago, the dams are all of earth or masonry. But a pump house, connected with the Manchester Ship Canal, has been built of reinforced concrete. It contains a large Diesel engine and a 36-in. centrifugal pump, and is notably free of vibration. There are also some reinforced concrete culverts in connection with the same work.

A reinforced concrete pontoon, 100 ft. long and 28 ft. wide, now in process of construction, is referred to.

Sundry Works.—Mr. Vawdry states that a number of other applications of reinforced concrete have been made in connection with navigation works, such as guide piles, sheet piles, coal tipples, coaling stations, and guide booms, some of the last-named being extremely heavy.

In connection with warehouses, Mr. Vawdrey describes some of the limitations of design imposed by municipal authorities upon reinforced concrete buildings, noting especially the requirements that columns in lower stories of warehouses must be designed for all floors fully loaded, and that exterior walls must be heavy enough for bearing walls.

While Mr. Vawdrey submits no formal conclusions, it is apparent that he considers reinforced concrete a suitable material for use in connection with navigation works.

PAPER BY HUNGARIAN STATE WATER SURVEY.

In this paper, submitted by the Hungarian State Water Survey, a dam and two locks are described.

A dam and lock at the mouth of the Körös River at Bökeny were built entirely of reinforced concrete. The dam is of the Poirée type, 35 metres long across the current and 21·35 metres wide in the direction of the current, with a difference of level between pools of 3 metres.

The invert is protected against undermining by reinforced concrete sheet piling driven on the upstream side. The driving was done partly by water jet and partly by a hammer. Longitudinal grooves in the sides of piles afforded means of closing the joints between them, the grooves being filled with cement.

The invert and the piers of the dam were built in reinforced concrete piles with circumferential grooves, designed to increase the resistance to pulling; the invert was fastened down to the piles so that any tendency to lift, due to hydrostatic pressure from the upper pool, would be resisted by the piles themselves, as well as the weight of the masonry. The invert consisted of principal and secondary beams, connected by a relatively thin slab; the pockets thus formed are filled with lean concrete in mass. The side walls of the dam were built with ribs. A foundation of mass concrete, from ·20 to ·25 metre thick, was put in over the entire site, to serve as a level foundation for the reinforced concrete work.

In the main, light steel members were used for fastening the piles and the invert together.

The Bökeny lock is 70 metres long, 10 metres wide, with a lift of 3 metres. It is not built in or along the river, but in a canal, cutting off a bend. As the soil here was firmer than at the dam, the lock is not built on piles, but the invert was made stronger. It consists of principal and secondary beams, with the pockets filled with lean concrete, as in the case of the dam. The invert was calculated as a girder, resting on an elastic foundation, and carrying a uniformly distributed load. Some details of the assumption made in the calculation of the invert are set forth in the paper.

The side walls are built with ribs, connected to the invert beams, so as to make a monolithic structure of the whole lock. No masonry facing is used on this lock, but the sills are protected with angle-irons or plates, and the side walls are protected by vertical timbers, fastened to the ribs.

In connection with both the dam and the lock, watertight inverts and bank revetments of reinforced concrete have been constructed both above and below, to direct the flow of the water and provide additional safeguards against scouring. The invert and revetments consist of reinforced concrete slabs, fastened down with small reinforced concrete piles at the corners, the joints between the slabs being afterwards filled with concrete.

This lock and dam have been successful. The paper discusses the advantages of reinforced concrete for such work erected on wet and compressible foundations. It is pointed out that a reinforced concrete invert is only from $\frac{1}{3}$ to $\frac{1}{2}$ as thick as an equivalent invert of mass concrete. This reduces the depth of the excavation, lessens the trouble with the ground water, and thus does away altogether with a large part of the most expensive work.

Reinforced concrete can also accommodate itself to slight inequalities of settlement without cracking.

The opinion is expressed that the consistent use of intersecting beams, forming rectangular pockets, and the adoption of light reinforcement conforming to a complicated design, as in the case of the work at Bökeny, are not the best methods for general use, on the ground that they are not the most simple in execution.

Some cracks have appeared in the side walls of the lock, which are interpreted as an indication that expansion joints ought to be provided; the absence of a facing better adapted than concrete to resist abrasion is also the cause of some doubts as to the future.

The Hungarian paper also describes a lock in course of construction in one of the branches of the Danube, near Budapest. It is founded on stiff clay, overlying a bed of coarse gravel. A competition was held for the design of this lock, and while one plan was adopted and the work entrusted to the designer, several of the competitive designs were acquired, and some features taken from them are embodied in the final working plans. One of the competitive designs is described in some detail and is noteworthy for the full advantage taken of the structural value of reinforced concrete in securing side walls of minimum weight.

In the adopted design the site of the lock was surrounded by caissons, which were apparently of timber, filled with concrete, with a small amount of reinforcement. The caissons were sunk by compressed air.

The following features were embodied in the design:—

1. Elimination, as far as possible, of the consequences of unequal settlement.

2. Employment of reinforced concrete only where its use is justified by lack of space, or to avoid heavy loads.

3. Protection of the side walls and the heads of the lock by facing.

The cross section of the lock was divided into three parts—namely, two self-supporting retaining walls, with spread footings forming a portion of the invert, and a separate section of invert, filling the space between the edges of the wall footings. The joints were closed by means of corrugated copper sheets.

A continuous reinforced invert is used at the heads of the lock. The concrete of the side walls is tied to that of the caissons by means of rods embedded in both.

The caissons, sunk into the clay, are depended upon to render the lock watertight.

The opinion is expressed that reinforced concrete is especially suitable for use where the foundations are weak and heavy loads dangerous.

The inference is that, in other situations, more massive construction, without reinforcement, is probably to be preferred.

No formal conclusion is presented with the Hungarian paper.

PAPER BY M. M. PERILLI.

This paper, by M. Mederico Perilli, describes a number of interesting and important works in reinforced concrete in various parts of Italy. The descriptions are accompanied with numerous illustrations and with general cost figures in a number of cases.

M. Perilli states that the applications of reinforced concrete to hydraulic works in Italy are very numerous, although those pertaining strictly to inland navigation are, as yet, relatively few.

. ***Works of General Interest.***—1. Resurrection bridge over the Tiber at Rome. This is an old bridge, with a span of 100 metres and a rise of 10 metres. The time required for its construction was only a little over twelve months. The foundations for the abutments had to be built in very poor soil. The abutments were supported on piles put in by a special process, which resulted in compressing the soil; pits for the reception of the piles were formed by dropping hammers of special form, and the concrete was rammed into them by the same means. Mass concrete was placed over the piles, up to ground level, and the abutments were built on this concrete in the form of caissons subdivided and strengthened by ribs in both directions. The arch is monolithic, 20 centimetres thick at the crown, and 50 centimetres thick at the springing lines.

It carries seven spandrel walls, on which the deck and superstructure are built. The total cost of the bridge was 1,250,000 lire.

2. Bridges over the Padrongianus and Posada torrents, in Sardinia. These are two bridges, one of seven and one of nine spans of 20 metres each, over torrential streams. They are continuous girder bridges, with four main girders to each span. The secondary beams form ribs for the floor slab, which is designed as a continuous T-beam.

The piers and abutments are built on concrete piles, reinforced with longitudinal rods and spiral hooping. The tests of all these bridges were eminently satisfactory.

3. Bridge of five spans over the river Magra. This is a graceful arch bridge built near St. Stefano. Each span is 50 metres, with a rise of 5 metres. The arches are supported on piers and abutments put in by means of compressed air. The gravelly subsoil under the piers and abutments was consolidated by the injection of cement grout under pressure.

The arches are 6 metres wide over all, but the deck slab overhangs and has a width of 7·2 metres over all. This bridge was subjected to severe test loads and carried them satisfactorily.

4. Composite bridge on the Fosso Vecchio, in connection with improvement works on the Reno.

This bridge consists of trussed girders, in which the tension members are of steel, while the compression members are of reinforced concrete. The reinforcement is fastened to the steel members. The reinforced concrete floor system takes the place of the floor girders and beams as well as the lateral bracing required at the floor level of an all-steel bridge. This bridge carried its test loads in a satisfactory manner.

5. Syphons under the branch of the Omborne, in the province of Grosseto. These are reinforced concrete syphons for carrying away the waters from adjacent low lands.

6. Weir over a network of rivers in the Province of Ravenna. This is a movable dam of the needle type, designed to control sediment-bearing waters which are distributed by various works over adjacent low lands, for the fertilising value of the sediment. The needles are supported on revolving iron bars, which in turn rest on fixed trestles of reinforced concrete. The sills and trestles are supported by piles, apparently of reinforced concrete, though this is not specifically stated. Infiltration under the sills is prevented by sheet piling, also apparently of reinforced concrete. By a special device, all of the needles can be raised from the sill simultaneously; they then swing down stream under the influence of the current, thus opening the dam almost instantaneously.

II. Works pertaining to Inland Navigation.— 1. Protective works on the banks of navigable canals.

The sandy banks of the Candiano Canal, in the Province of Ravenna, have been protected in places by means of reinforced concrete piles, about ·2 metre square, driven 3 metres apart, centre to centre, connected at the top by a beam of the same cross section as the piles; behind the beam the space is closed by sheet piling.

The length of the piles varies according to circumstances, and where their length is great they are anchored back to shorter piles driven some distance behind them in the solid bank.

At some points sloping banks have been riveted with reinforced concrete slabs, which simply extend up from the toe of the bank, with the edge at the toe protected by rip rap; in some cases the revetment abuts at its lower edge against a concrete foundation wall deep enough to prevent scour; in other cases, where the water could not be drawn off, the revetment springs from a row of piles.

2. Unloading wharves along the Canal of Porto Corsino, in the Province of Ravenna.

Some extensions of loading and unloading wharves of this canal were executed in reinforced concrete, as follows:— .

A row of ground piles is driven along the water edge. Two parallel rows of piles are driven in the rear, making three rows in all, 3 metres, centre to centre. The three rows of piles support a system of beams and a reinforced concrete slab; on the water side of the superstructure is a longitudinal wall, 1·73 metres high; behind this the platform is covered with earth, which is depended upon to increase the stability of the work.

3. Landing stages on the Baiona Canal of Ravenna.

These are reinforced concrete wharves designed for mooring vessels and for handling coal. They consist of reinforced concrete piles, carrying a system of trussed girders and a floor slab, all of reinforced concrete. The details are well shown in the illustrations.

4. River port on the Tiber, at Rome.

This work, undertaken to increase the water-borne traffic, is designed to regulate the river and finish the banks in such a manner as to facilitate the handling of merchandise.

The low-water channel is confined between rows of reinforced concrete caissons, resting on a foundation of rubble stone, levelled by divers; the caissons are filled with mass concrete. Some of the caissons were built of concrete blocks, the reinforcement being built in. Others were cast in moulds. They were built on shore and floated into place. On the shore side of the caissons is a gently sloping berme of concrete, covered on top with a slab, apparently of reinforced concrete. The berme is succeeded by a slope of 45°, heavily riveted to a level above high water. A total of 500 metres of these quays have been built. They have proven satisfactory.

5. River port on the Po at Pontelagoscuro.

This work consists essentially of reinforced concrete wharves for handling freight. It is in two sections, one for light traffic, and one, equipped with cranes, for heavy traffic.

Both sections consist of reinforced concrete columns, resting on reinforced concrete piles, with the usual and necessary tie beams and braces and a floor system, all of reinforced concrete, the whole constituting a framed structure. Each column rests on a group of four piles.

6. Station on the Tiber for unloading coal for the Anglo-Roman Company.

This consists essentially of a crane track for the unloading crane. Two rows of octagonal piles of hooped concrete carry longitudinal beams, which support the crane track; the tops of the piles are also connected by cross beams and diagonal braces of reinforced concrete.

7. Boats. Several types of boats are described, all consisting of metal framework bedded in concrete.

A 50-ton barge for carrying merchandise, and a wagon ferry, are illustrated. Several others are briefly described. Some of these vessels have been tested under both load and impact and then accepted; it is inferred that they are successful.

8. Syphons for regulating water level.

Two forms of syphon, designed to regulate the level of the water in a reservoir, or upper pool, are described. Their interest lies in the fact that they are built almost entirely in reinforced concrete.

M. Perilli says the conclusion is justified that reinforced concrete has technical and economic advantages which adapt it for use, and entitle it to preference in many cases.

PAPER BY MR. RICHARD L. HUMPHREY.

Mr. Humphrey gives some details of the origin of hydraulic cement, both "Natural" and Portland. He also gives some facts of interest relative to early canals in the United States, showing how the requirements of lock masonry gave rise to a demand for a cement which would set under water. This resulted in the invention and manufacture of cements of the "natural" or Rosendale type which until recently were widely used in this country.

A *résumé* is given of the qualities which render concrete so generally useful, and reference is made to important works of recent date, which probably would have been prohibitive in cost but for the availability of Portland cement, which, in one case—the Roosevelt dam—was manufactured on the site from local materials with satisfactory results. It is stated that the annual consumption of Portland cement in the United States amounts to 76,000,000 barrels, and that much of this is used in connection with waterways. Some reference is made to the danger of deterioration where concrete is exposed to freezing in contact with water, or to water highly charged with alkaline salts; and to the general opinion that rich, dense, well-mixed concrete will withstand these agencies.

The author then proceeds to cite many cases of the application of concrete, both plain and reinforced, with the object of giving an idea of the extent of this application, rather than giving details, alhough the latter are not wholly lacking.

Canals. — Under this heading the author refers to a number of important works, and submits photographs which largely speak for themselves. The works referred to are the Panama Canal, the New York Barge Canal, several works of the Corps of Engineers, U.S. Army, and one executed by the American Steel and Wire Company on the Monongahela River.

Locks and Dams. — Under this heading the author submits views and brief data relative to a number of locks and dams in the United States. He also refers to the use of concrete for bank protection and for breakwaters.

A reinforced concrete retaining wall at Columbus, Ohio, is illustrated. It is of light section, and resists the pressure by transmitting the thrust through buttresses or counterforts to the foundation.

A bank protection of concrete blocks at Junction City, Kansas, a heavy massive retaining wall built by the National Tube Works along the Monongahela River, and the Galveston Sea Wall are all illustrated and briefly described.

Quay wells and sea walls of various types are shown, and a reinforced concrete shell for protecting timber piles from the toredo is illustrated by photographs.

Harbour Works. —This section of the paper is introduced with the statement that concrete has come to be a standard material, not only for quay walls, but also for piers, docks, and other structures, and that the reinforced concrete pile is gradually taking the place of the wooden pile because of its superior durability. It is also stated that, while a reinforced concrete pile is more expensive than a wooden one, a smaller number is required, so that where the upper ends of the piles are built into a concrete superstructure the total cost is often less than with wooden piles. The reinforced concrete piles are generally moulded on shore, then either driven or jetted into position; sometimes both driving and jetting are resorted to. It is stated that piles driven after hardening are better than those moulded in place inside of a casing, because the casing is usually not watertight and the access of the water to the green concrete prevents the latter from being of the best quality.

Irrigation Works. — The extensive works built and under construction by the

United States Reclamation Service afford many illustrations of the success and economy attending the use of concrete. The Roosevelt dam, where the cement was manufactured on the site, is one example.

Miscellaneous.— Under this heading attention is again called to the use of reinforced concrete caissons made on shore and floated into place, as at the Algoma Breakwater; also to a method of depositing concrete under water through a tremie pipe with its lower end buried in the concrete; the pipe is raised as the concrete flows out at the bottom, but is kept buried in the mass, so that only the outer layer of concrete is in contact with the water. This method was used in constructing the Detroit River Tunnels.

Destructive Agencies.—Under this heading Mr. Humphrey discusses the various kinds of distintegration that have appeared in concrete structures. He reaches the conclusion that poor materials or workmanship, or mistaken methods, are at the bottom of most of the trouble. He believes that, in case of concrete disintegrating in either fresh or salt water, the trouble is due to porosity and freezing, and states that many engineers have expressed similar views. He thinks that sea water has but little chemical effect upon concrete, and then only after disintegration by freezing has begun. Nevertheless, he recommends the use of enough silica in proper form to satisfy all the free lime in cement that is to be exposed to sea water.

But he considers that the chief cure for the trouble is to select and proportion the materials so as to secure a dense impervious mixture, and to allow the concrete to harden before exposure.

He recommends a reasonably wet mixture—as wet as possible, to avoid danger of separating the ingredients—and very thorough mixing. The aggregate must be hard and dense, and chemically inert, and every precaution taken to ensure a homogeneous and impervious concrete. Such material, if permitted to harden before exposure to sea water, may be expected to resist its destructive effects, as well as the action of the first. Mr. Humphrey states, as a result of his own observation, that even rather poor concrete shows little evidence of deterioration below low water line; he also states that in tropical waters no good concrete has suffered, but does not give specific examples.

Mr. Humphrey also discusses the disintegration that has appeared in concrete structures erected by the Reclamation Service in the alkali regions of the west. The trouble is attributed to the crystallisation of alkaline salts in the concrete, producing a mechanical effect similar to that produced by freezing. The alkaline ground water is absorbed into the concrete by capillary action and evaporates from the surfaces exposed to the dry air; this results in the formation of crystals in the concrete, with a consequent disruptive action. Similar trouble has been experienced under the same conditions with other building materials, such as stone and bricks, which were porous enough to absorb any considerable percentage of water. Mr. Humphrey considers that density and strength are the best safeguards against this action, but states that the problem is not completely solved.

He refers to some experiments recently conducted by the Department of Agriculture that indicate that the addition of 5 to 10 per cent. of residuum oil of petroleum may result in the desired degree of imperviousness.

The dangers due to corrosion of the embedded metal in reinforced concrete are discussed and illustrated by references to some reinforced concrete piles in Atlantic City, where, due to the use of porous aggregates and to imperfect covering of the reinforcement, the metal has exhibited progressive corrosion which promises soon to destroy the work. Other reinforced concrete piles in the same locality, in which better materials, methods, and workmanship were employed, have shown no signs of deterioration.

Mr. Humphrey submits seven conclusions, which may be summarised as follows :—

1. Concrete, because of its durability, fire resistive properties, compressive strength, plasticity in the green state, and the wide distribution of available aggregates, has proved to be "one of the most economical of building materials."

2. Plain concrete in mass is always suitable where the stresses are principally compressive.

3. By suitable reinforcement, concrete is enabled to withstand tensile stresses, and becomes suitable for use in a much wider field.

4. Concrete suitably mixed, placed, and reinforced is especially suitable for watertight structures, and when "mixed and placed so as to secure a mass of maximum density it is in itself watertight, and the addition of liquids, powders, or coatings for increasing its watertightness is unnecessary."

5. Its durability and low maintenance cost make it specially economical in permanent structures.

6. While opinions differ as to its durability in fresh and sea water, concrete mixed and placed so that the resulting mass is of maximum density affords ample resistance to the action of both fresh and sea water, especially if allowed to harden before exposure.

The consensus of opinion on this point seems to be as follows :—

(*a*) "Little if any damage is done to even the poorest concrete structure below water."

(*b*) "The principal destructive actions occur between tides, generally at the mean-tide line."

(*c*) "Where the concrete has disintegrated in sea water this is undoubtedly caused by freezing, for the reason that in tropical waters where there is no freezing the structures do not show this deterioration, excepting in cases where sea water has been used in the mixing and the concrete has been placed through sea water, thereby producing an inferior structure unsuited to resist sea water action."

(*d*) "While there may be some chemical action after the concrete is weakened through freezing, it is a fact that there is little or no chemical action on a properly proportioned, mixed, and placed concrete."

(*e*) "It seems to be a fact that it is desirable that the cement used in making concrete to be exposed to sea water shall contain sufficient silicious materials as will satisfy whatever excess lime there may be in the hardened cement. This is accomplished in many parts of Europe through the addition of trass or puzzolana to hydraulic cement containing an excess of lime, which tends to increase its strength and render it stable in sea water."

(*f*) "It is quite apparent that one of the prime essentials for a concrete structure that will be immune against sea water action is that the surface shall be dense and impervious. An attempt has been made to secure this condition by applying mortar under an air pressure of upwards of 30 lb., and while this undoubtedly increases the density as compared to hand methods, nevertheless it remains to be seen whether it achieves the object desired."

(*g*) "Good practice demands that the concrete shall be mixed a sufficient length of time, without too much water, so that there results a mass of viscous consistency which will flow readily and yet the ingredients not separate. If such a concrete is deposited under conditions which will prevent the sea water permeating it before it has set, such concrete affords excellent resistance to sea water. Another method proposed has been to deposit the concrete in tremie, as with this form of construction only the upper surface comes in contact with the water, and there results a concrete which is not affected by sea water action."

(*h*) "When reinforced concrete is used in sea water it is essential that the aggregate shall be a hard, dense material of low absorption, and that the reinforcement be protected by a coating of at least one inch of silicious mortar."

7. When concrete is subjected to the action of alkaline or acid waters the same precautions must be observed as in the case of exposure to sea water. "Density and strength are the two prime essentials for adequate resistance to such action."

The increasing use of concrete and its existing record of successful results justify the conclusion that it is peculiarly adapted for use in connection with hydraulic works.

REMARKS BY THE GENERAL REPORTER (Mr. John Stephen Sewell).

In all the papers reviewed the authors have condensed a great deal of information into a very small space. A summary which attempted to present more than an indication of the nature of their contents could not well be made in less space than that of all the papers combined.

It is quite apparent that the use of concrete, both plain and reinforced, for all classes of hydraulic work is making rapid headway, especially in the United States, France, and Italy; and that opposition to its use is disappearing elsewhere.

Strictly speaking, the application of reinforced concrete to hydraulic structures is the only subject under discussion. All of the objections to the use of plain concrete in such cases are included within the greater number that have been urged against reinforced concrete. What follows, therefore, will apply especially to the latter material, and to the conditions surrounding its use in connection with works of inland navigation.

The objections that have been raised against the use of reinforced concrete for the purpose under discussion relate to the durability of the concrete itself, to its resistance to abrasion, chemical action, and freezing in contact with water, and to the durability of the reinforcement under various conditions.

Durability of Concrete.—A few years ago difficulty was sometimes experienced in securing a Portland cement that was sound and secure against disintegration due to chemical changes within itself after setting. This difficulty is now easily avoided, and if aggregates are selected of durable and inert materials there can no longer be any doubt that concrete is in itself a thoroughly durable material, quite secure against disintegration due to action originating within the mass itself.

Resistance to Abrasion.—Concrete, as a rule, exhibits more chipping and chafing under impact and abrasion than masonry of the harder stones. But this damage is usually only superficial, and rarely threatens the integrity or continued usefulness of the structure. In many cases it is more resistant that any other form of masonry available within practicable cost limits; the best of masonry is liable to be disfigured under impact and chafing, and there is very little of it in existence subject to these conditions that does not show the wear and tear. In any case, the trouble can be avoided at moderate expense by means of strips of steel or timbers applied in the proper manner. The objection to concrete is therefore not a valid one, for it can be removed by simple and practicable methods.

Resistance to Chemical Action.—Under this head may be included atmospheric agencies, the action of sea water, of sewage, and of acidulated water. So far as atmospheric agencies are concerned, there is too much concrete which has successfully withstood them to leave any room for discussion. It is merely a question of good materials and workmanship, including proper mixtures to secure a dense and impervious mass.

A great many concrete structures exposed to sea water have suffered from extensive and rapid disintegration. It was at first supposed that this was due to the action of the sea water on some constituent of the cement, probably the free lime. Some experiments and investigations have seemed to indicate that the addition of trass or puzzolana to the cement would prevent this action by satisfying the free lime. But it also appears probable that, as Mr. Humphrey states, a dense, strong, and impervious mixture, allowed to harden before exposure in place, is in itself sufficiently resistant, whether the preliminary disintegration has been due to freezing when saturated with water or whether it is due entirely to chemical action. The requisite density and strength can best be obtained with a well-balanced and rather wet mixture.

There is no doubt that wet mixtures will give denser concretes than dry ones, no matter how thoroughly the latter may be rammed. The use of dry mixtures had its origin, no doubt, in laboratory experiments—such as those of the late General Gilmore, of the United States Corps of Engineers—which indicated that dry mixtures, when well rammed, produced cubes of greater strength than wet mixtures having the same proportion of cement and the same aggregates.

The use of wet mixtures originated in this country, at least, from the fact that they require little ramming, make a smoother finish against forms, and generally cost less in place than dry mixtures well rammed. The practice of using wet mixtures was initiated by contractors, who were actuated by motives of economy, and made its way slowly against the opposition of engineers. It is fortunate in this case that the cheapest method is also the best; but it is only within a few years at the most that engineers have been fully convinced. Mr. Humphrey's warning against an amount of water sufficient to separate the ingredients should not, however, be overlooked.

It seems certain that there are many cases of concrete structures which are exposed to sea water in both warm and cold climates, and which are behaving satisfacfactorily. One successful example is of more value as evidence than a hundred failures; for it proves that success can be attained. If the necessary precautions are not yet thoroughly understood, it is only a question of further trial and study when they will be. The conclusion seems justified that exposure to sea water is not necessarily fatal to the use of concrete. As inland navigation may begin before the salt water ends this point is germane to the subject under discussion.

Navigation canals may carry domestic sewage, factory waste, and acidulated water from mines, since all of these ingredients are to be found in the waters of streams from which canals are fed. So far as navigation works are concerned, the deleterious ingredients will generally be so diluted that their action will be very slow, and probably the same precautions that suffice in the case of sea water will serve the purpose here also. Should cases arise where the deleterious ingredients exist in greater proportion it is probable that any kind of masonry would suffer more or less by action upon the cement in the joints if in no other way. Limestone is probably more commonly used than any other natural stone for hydraulic works, and it would suffer in much the same way as cement. Many cases will arise, also, where there is no suitable stone within easy reach, and if concrete is not used the work may be impracticable. M. Jacquinot describes a reinforced concrete conduit for carrying sewage; many large sewers of reinforced concrete have been constructed in the United States. The inverts are sometimes protected by other masonry—often of vitrified bricks or blocks—and sometimes by pitch or other coating. Many reinforced concrete chimneys have also been built without a lining, and they seem to resist the chemical action of the flue gases, and, in some cases, of the gases from smelters, very well. Melted paraffin could be applied to the finished surface and driven in by heat; while it is not known that this method has been tried, it is known to be inexpensive, and should greatly increase the resistance to acids. Finally, there are many methods of waterproofing masonry, any one of which ought to protect the concrete from contact with acidulated water or sewage, and therefore from damage. It appears, therefore, that even the existence of these agencies need not be a fatal objection to the use of concrete, for their activities can be prevented at practicable cost; only in extreme cases would such protection be required in navigation works, and even then the protected concrete may easily be the least expensive material available.

Freezing in Contact with Water.—Resistance to damage from this cause seems to be merely a question of density and strength. The same thing is true of stone and bricks. That concrete can be made sufficiently impervious and strong is demonstrated by many examples, such as the reservoir works cited by M. Jacquinot, and numerous structures in the northern part of the United States.

Here, again, a well-balanced wet mixture, protected from washing out of the cement during the hardening process, is all that is required.

The damage from waters charged with alkaline salts seems to be of a mechanical nature, similar to that produced by freezing. While, as Mr. Humphrey states, the problem here presented is not completely understood or solved, it seems probable that it is again merely a question of sufficient strength and density. While the problem is in process of solution, however, protective coatings are still within reach at moderate cost, and there is no valid objection to the use of concrete with proper precautions.

Durability of Reinforcement.—It is no longer open to serious doubt that steel or iron thoroughly embedded in Portland cement concrete will last indefinitely as long as the covering remains intact. If it is exposed directly to the air, whether near the sea or not, it will inevitably corrode and ultimately destroy the structure. Careful design and good workmanship are all that are required to properly embed it in the first place. The only danger that threatens it thereafter is the danger of cracks, which will destroy the integrity of the concrete and open up a way for atmospheric moisture or water to gain direct access to the reinforcement. Such cracks might be due to shrinkage in setting, to expansion and contraction under changes of temperature, or to deformation under stress.

If the concrete is mixed wet and kept wet while setting there is small danger of shrinkage cracks. In fact, cracks of this kind are rarely seen in practice. Light reinforcement is sometimes used with a view to preventing them, but it is not easy to see how it would be effective, for the shrinkage of the concrete would set up compressive stresses in the reinforcement, and it is not usually heavy enough to resist them effectively. However, the danger is more imaginary than real, and an abundant supply of water and protection from evaporation during setting will ensure a successful result. If the concrete is reinforced for serious structural duty the reinforcement will generally be heavy enough to prevent shrinkage cracks, even if they tend to form in exceptional cases.

Cracks due to expansion and contraction after setting are brought about probably by a slight slipping of the mass on its bed during expansion, and by the excess of frictional resistance over the tensile strength of the concrete during the subsequent contraction. This trouble can be overcome by proper reinforcement, but it would be well to divide a long wall or other structure into sections, so that each could act as a unit. The joints can be cared for in some such way as described in the paper of M. Jacquinot and in that of the Hungarian State Water Survey.

Cracks due to deformation under stress will occur only when the reinforcement is stressed so that the strain exceeds the limit of extensibility of the concrete. This can be avoided by proper design and workmanship. This danger makes it inadvisable to utilise the high working stresses, otherwise permissible in high carbon steel, since the modulus of elasticity is no greater than with low carbon or medium steel. If working stresses are kept well within the limits allowable for mild steel there is no danger of cracks in the concrete.

It appears, therefore, that it is entirely possible to maintain the integrity of the concrete coating, and therefore to prevent corrosion of the reinforcement, and there is no valid objection on this score.

Some laboratory experiments have caused some engineers to fear that reinforced concrete subjected to hydraulic pressure will have the bond between the concrete and the reinforcement destroyed, so that the two materials would cease to act in unison. But the old water tank cited by M. Jacquinot is in itself sufficient to settle this question favourably, even if there were not available the evidence of the numerous conduits sustaining high pressures cited by the same author, and the evidence of similar structures in this country.

Mr. Humphrey refers to the corrosion of reinforcement exposed near the sea. It is also a fact that if the concrete is mixed with sea water, or with sand from the sea beaches, or if it has salt mixed with it, and it is subsequently exposed to dampness, the reinforcements will corrode. This was shown by examples on a rather extended scale in some fortification work on an island in Long Island Sound some years ago. The General Reporter, then in the service of the United States, inspected this damaged work, and afterwards had two experimental slabs of reinforced concrete made with a view of finally testing the matter. They were of identical composition except that one was mixed with sea water and one with fresh water. They were exposed to the weather on a roof in Washington, D.C., for some months. At the end of that time the reinforcement in the sea water slab was badly corroded, while that in the other was entirely untouched. It is of importance, therefore, that the ingredients used in mixing concrete for hydraulic works should contain no corrosive material in themselves if the concrete is to be reinforced.

The conclusion seems justified that all of the objections that have been urged against the use of reinforced concrete as a material suitable for use in connection with hydraulic works are either imaginary or can be overcome by practicable methods, and must have arisen at a time when the subject was not so well understood as at present. That this conclusion is justified is abundantly proved by the increasing and successful use of the material in permanent structures everywhere, as indicated by the papers herein reviewed.

The great advantage of reinforced concrete lies in the fact that it is capable of withstanding stresses due to transverse strains, tension, and shearing. All the forms that could be executed in steel or timber can be closely imitated in reinforced concrete, which is immune from corrosion and decay. This makes it possible to adopt designs wherein the structure acts by its structural resistance and not by dead weight, and even the material to be retained and held back may be made by this means to add to the stability of the work as a whole. Dead weights on foundations are diminished, difficult excavation is often avoided or lessened, and total costs often greatly decreased, as compared with structures formed of masonry in mass; in many cases reinforced concrete affords the only practicable solution of a difficult problem, and in nearly all cases it affords a variety of desirable solutions not practicable in any other material.

The saving in thickness of inverts of locks and dams, or retaining walls of all kinds, the use of caissons filled with dead materials in lieu of solid masonry walls, the use of reinforced concrete piles to anchor a light structure to the dead mass below, and the many other useful devices and applications, either described or suggested in the papers on this subject, all open up the possibility of practically limitless applications of reinforced concrete to hydraulic structures so as to attain both greater efficiency and a diminished cost.

A study of the successful applications of reinforced concrete submitted to the Congress by the various reporters appears to justify the adoption of the following conclusions:—

Reinforced concrete combines the structural qualities of steel and timber with the durability of good masonry. It is subject to no form of deterioration which cannot be avoided by reasonable precautions. It is free from many of the limitations surrounding the use of masonry in mass; because of the greater latitude it affords in the design and execution of structures; it often yields the best and most economical solution, and in some cases the only practicable solution, of the most difficult problems.

When properly designed and executed it is, therefore, among the most valuable, if not the most valuable, material now available for use in connection with hydraulic works of all kinds.

CONCRETE TESTS DURING the ERECTION of the NEW WAR OFFICE in VIENNA.

By ENGINEER-LIEUTENANT
J. KROMUS

The account of the method of testing adopted during the erection of the new War Office in Vienna, which we publish below, is a summarised translation of a very useful article which appeared in "Beton und Eisen," and which has been issued in pamphlet form by Messrs. Wilhelm Ernst und Sohn, of Berlin.—ED.

THE new War Office buildings in Vienna are of unusually large size, and the attempt has been made to take the fullest advantage of modern methods in their construction. The site on the Stubenring is about 14,000 $m.^2$ in area (152,000 sq. ft.), whilst the area actually covered by the buildings is 9,700 $m.^2$ (104,000 sq. ft.). There are eight stories, including the basement, with an additional story in the middle section. The site being covered with a deep layer of filled-in material, it was necessary to make a deep excavation and to construct a mass concrete foundation, in which 27,000 $m.^3$ (35,000 cub. yds.) of concrete were used.

The Reinforced Concrete Work.—Reinforced concrete was used for the floors, the area of which amounted to 50,000 $m.^2$, of which 40,000 $m.^2$ (430,000 sq. ft.) were plate beam floors with flat under-surface, on the Porr system, the remainder being ordinary panelled floors. They were required to carry a working load varying in different parts from 300 to 1,000 kg./$m.^2$ (61 to 203 lb. per sq. ft.), the clear spans being as high as 9·25 m. (30 ft. 4 in.) and a certain definite clear height being required in all rooms. A further requirement was that the floors should be capable of carrying at each window column a brick partition, 15 cm. thick, without diminution of the clear height. Various beams, cantilevers, balconies, etc., were also constructed in reinforced

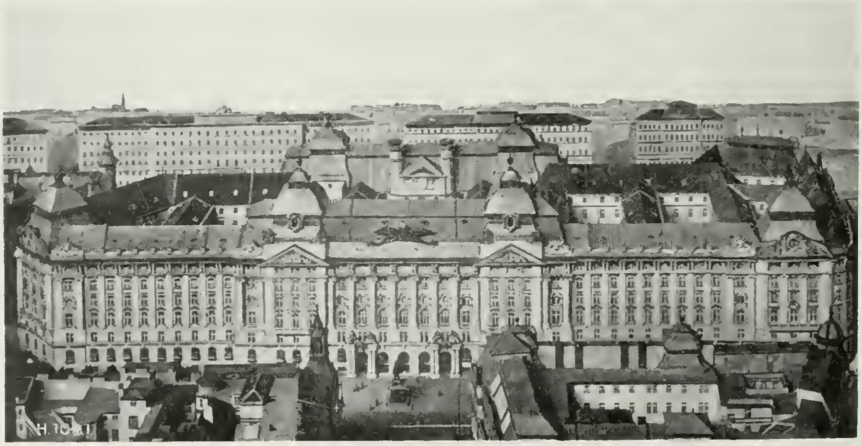


Fig. 1. General view of building.

CONCRETE TESTS DURING THE ERECTION OF THE NEW WAR OFFICE, VIENNA.

concrete. The roofs, however, were of wood. The reinforced work required 8,500 m.3 of concrete and 900 tons of steel. The reinforced concrete work was begun in July, 1910, and finished in September, 1911, after an interruption of three months during the winter.

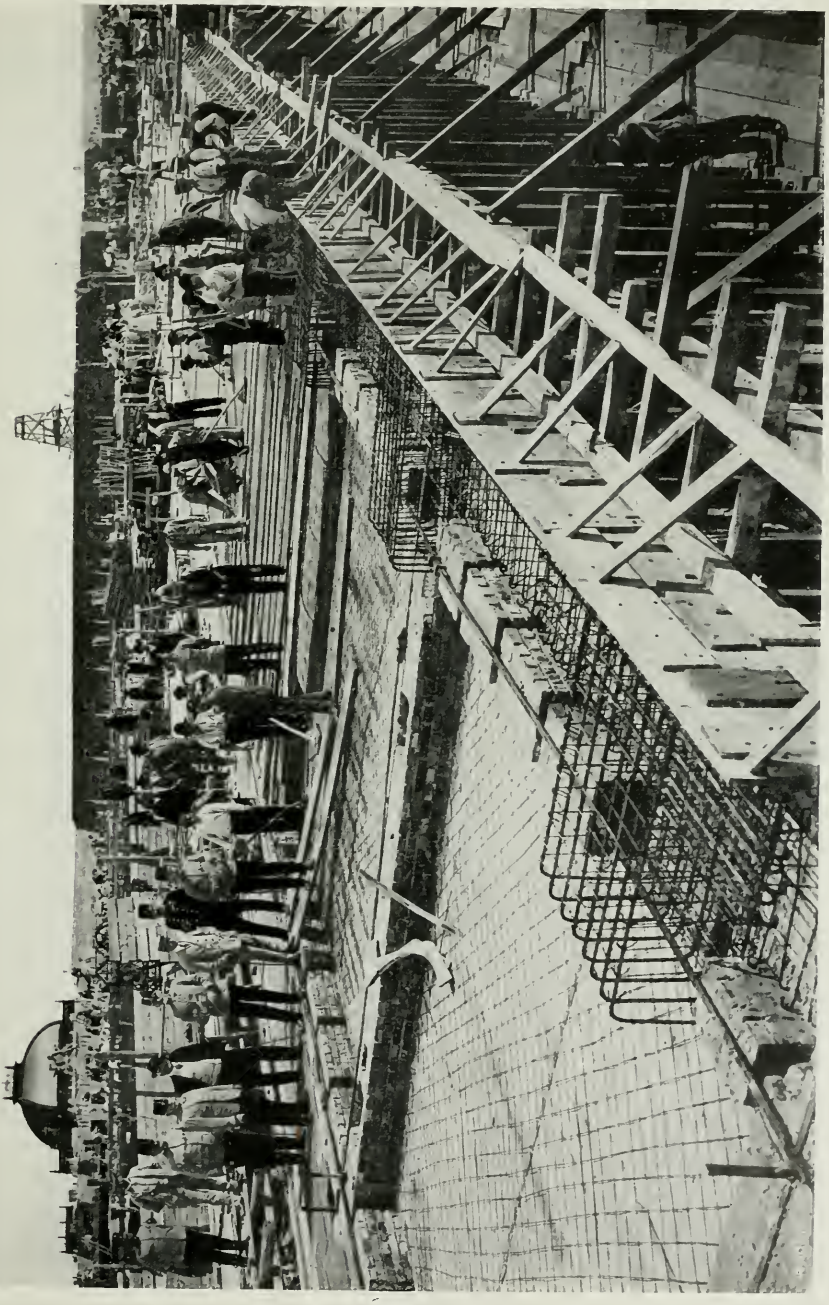


Fig. 2. Stages in the construction of a floor.

Concrete Tests during the Erection of the New War Office, Vienna.

Fig. 1 shows a general view of the building, whilst *Figs.* 2, 3 and 4 show different stages in the erection. *Fig.* 2 shows clearly the mode of construction of the floors.

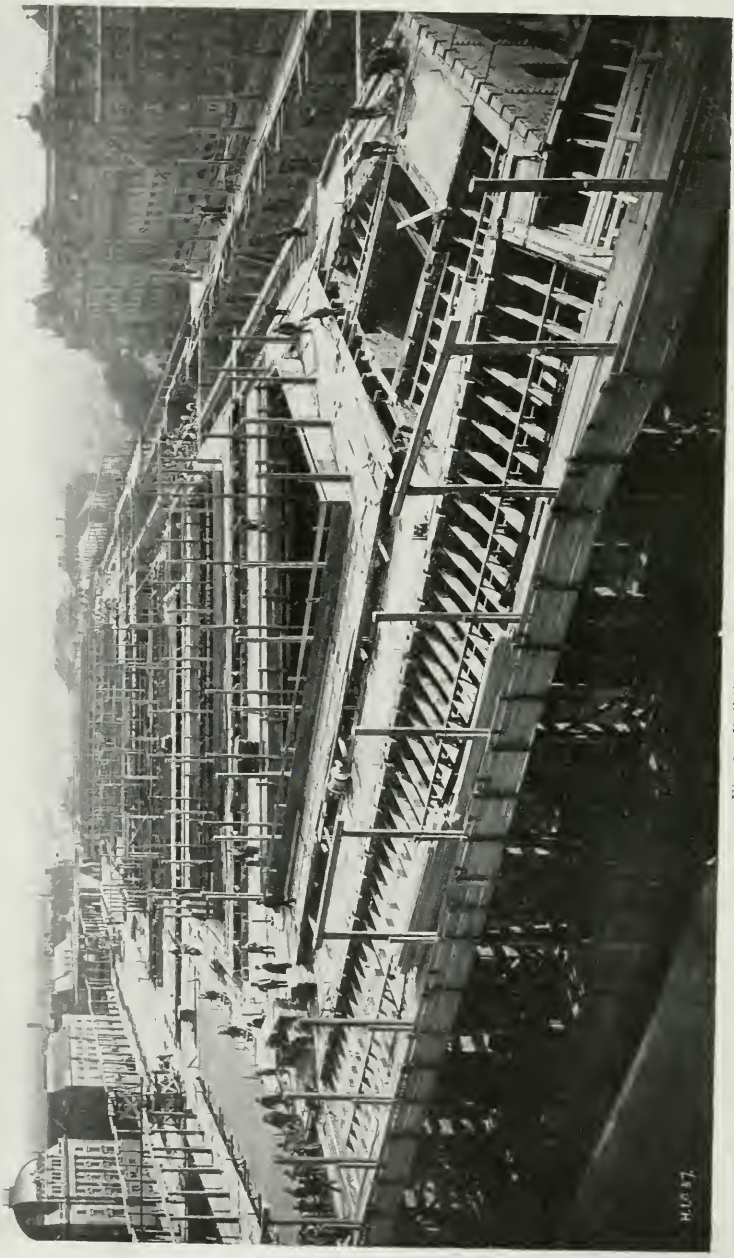


Fig. 3. Building at the level of the second story.

Concrete Tests during the Erection of the New War Office, Vienna.

Testing the Work during construction.—At the beginning of the work only the usual compression tests of concrete cubes were made, but on the publication in 1910 of Dr. F. von Emperger's method of Control beams (see Concrete and Constructional Engineering, 1911, p. 764) it was decided to adopt this system of testing, and the contractors expressed their willingness to

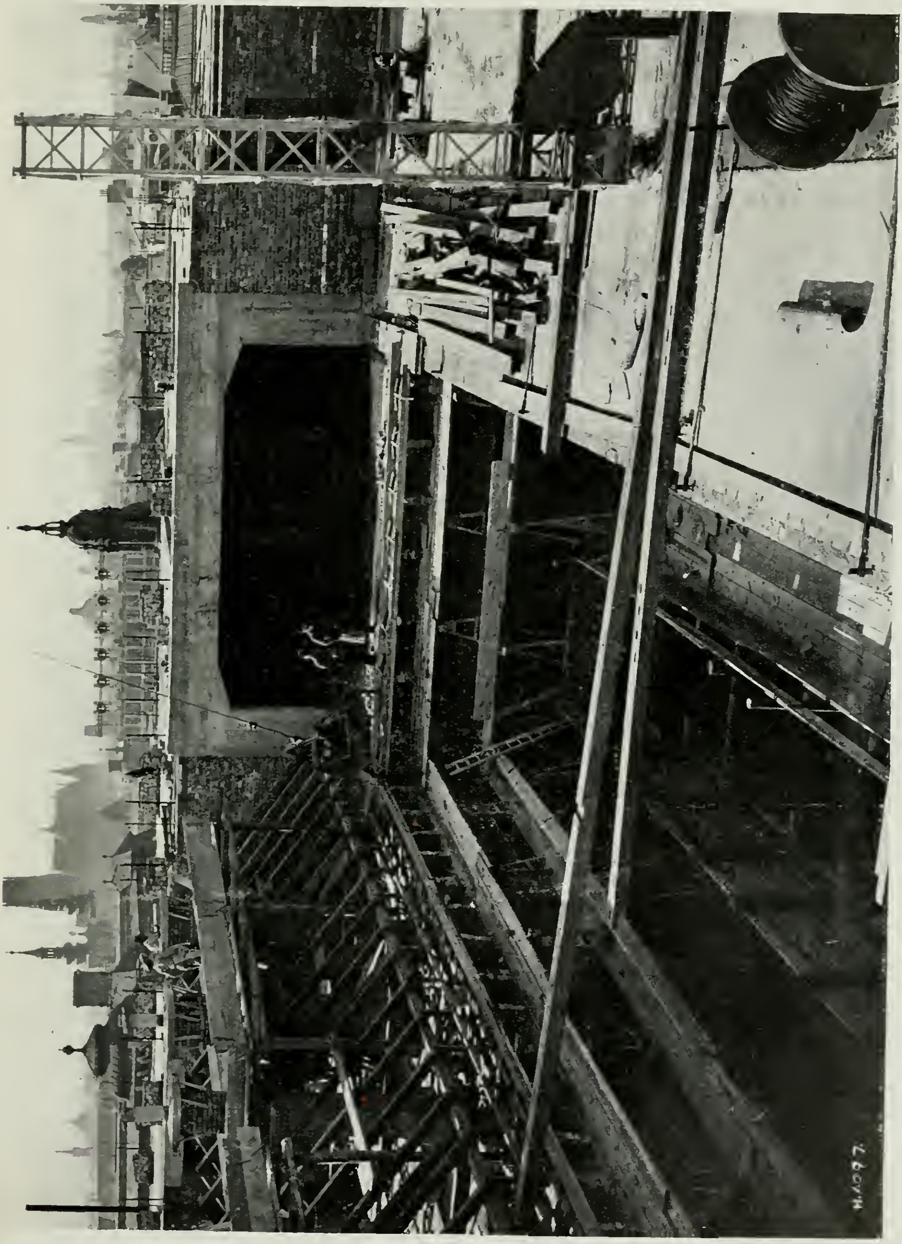


Fig. 4. Frame Construction in the Mansard Roof.
Concrete Tests during the Erection of the New War Office, Vienna.

prepare the necessary beams. Twelve pairs of beams were at first constructed, one beam of each pair having 2 per cent. and the other 4 per cent. reinforcement, whilst as a rule one pair was tested after fourteen days, and a second, exactly similar pair, after the winter period was over. In the course of the experiments the beams with only 2 per cent. reinforcement were found to be unnecessary, and subsequently only beams with 4 per cent. reinforcement were used. The total number of beams tested was seventy.

Performance of the Tests.—The beams were constructed on the spot, the moulds being held together by iron clamps, in order to ensure the accuracy of the dimensions. After three or four days the moulds were removed and the beams exposed to the same weather influences as the concrete in the building. Some damage was done to a few of the beams, owing to carelessness. The tests were performed exactly in the manner prescribed by Dr. von Emperger, the operations being all readily performed by ordinary workmen.

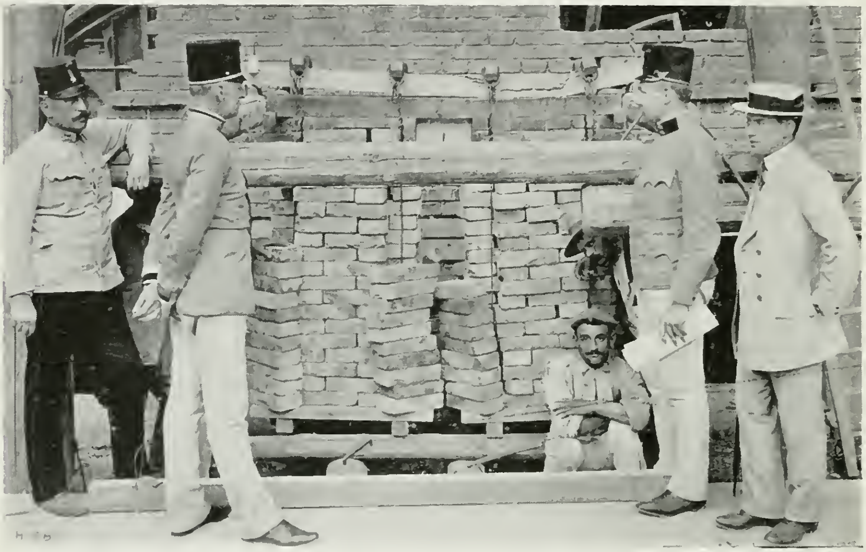


Fig. 5. Testing a Control Beam.
Concrete Tests during the Erection of the New War Office, Vienna.

Results of the Tests.—The most important factor in determining the results of the tests was the occurrence of frosts during construction and testing. For example, whilst during warmer weather the centering could be removed after ten days, as soon as night frosts had set in it was found that a beam only gave after ten days a strength of 98 kg./cm.^{2} (1,390 lb. per sq. in.). Beams prepared on a frosty day, but immediately covered with mats, showed a strength after fourteen days, and also after three months, of only one-half of the normal value. The broken surfaces showed the unmistakable markings of frost crystals.

In the second group of tests, after the severe frost was over, beams were prepared in such a way that at each time three tests with two pairs of beams and one test with four pairs were started, compression cubes being prepared for every fourth test. The beams were always tested after fourteen and twenty-eight days, and each fourth series also after six weeks. The curves expressing the results show very clearly that the conditions of temperature prevailing during the first few days after mixing the concrete are of the utmost importance, and that the bad influence of frost during this period is not to be remedied subsequently. The view formerly sometimes held, that frost merely retards the process of setting, is therefore untenable. The formation of ice-crystals brings about an actual structural alteration in the concrete.

With normal beams the strength attained is 310 kg./cm.^{2} after twenty-eight days, rising to 330 kg./cm.^{2} after six weeks, and 360 kg./cm.^{2} after six months (4,400, 4,700, and 5,120 lb. per sq. in.), whilst beams prepared at a low temperature did not reach more than 200 kg./cm.^{2}, and in one case only reached 165 kg./cm.^{2} (2,350 lb. per sq. in.) after six months. It must, of course, be considered that the small beams are more sensitive to such changes than larger masses, but this is not a disadvantage of the method of testing, which clearly indicates sources of danger. By drilling portions of the completed floors it was proved that the effect of frost was actually visible in the interior of those constructed at a low temperature. Protection with mats was found to be adequate. The cement and concrete proved to be very uniform in quality. The principal difficulty in the preparation of the beams was found to be the placing of the reinforcement, very small errors in this respect causing great loss of strength. Four exactly similar beams prepared and tested simultaneously gave breaking strengths differing only by ± 5 per cent. The corresponding tests with cubes differed by as much as ± 27 per cent.

If the normal strength found after fourteen days (250 kg./cm.^{2}, or 3,660 lb. per sq. in.) be called 100, the following short table expresses the principal results obtained :

Temperature during setting	14 days	28 days	6 weeks	6 months
Normal (over+5°c.) Max.	124	144	152	160
Mean	100	120	132	140
Min.	80	100	116	124
From 0° to+5°c.	64	82	92	—
0° and below	44	52	56	66

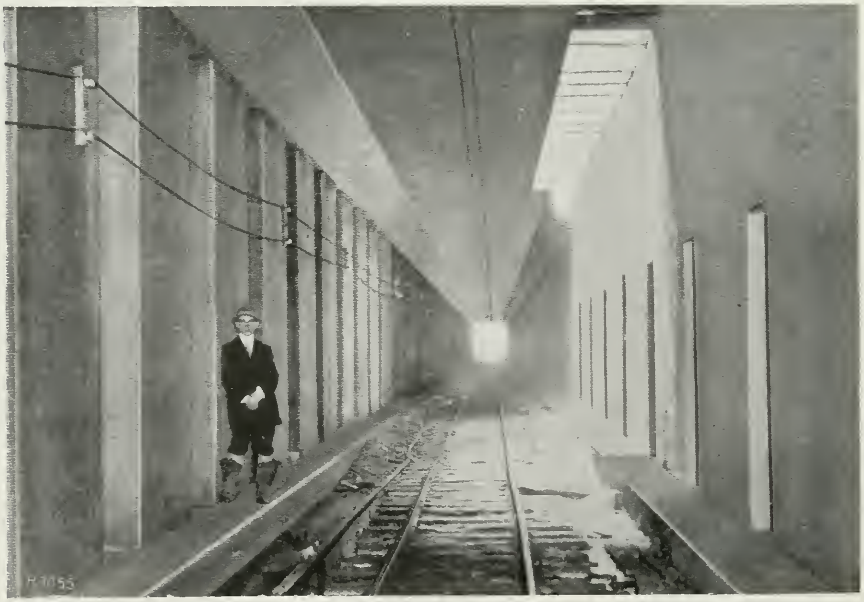
Mode of Rupture.—The beams usually broke by compression, and the cracks along the reinforcement, which often resulted in the splitting off of fragments of the concrete (see *Fig.* 5), were of secondary origin. Beams with 4 per cent. reinforcement did not show tension cracks until the tension in the steel reached 3,000 kg./cm.^{2} (427,000 lb. per sq. in.). This indicates that 4 per cent. is quite sufficient for most purposes.

The authorities express their entire satisfaction with Dr. von Emperger's system of testing, which affords complete and continuous control over the material, in a form which is readily understood and appreciated by contractors and workmen, and materially assists in maintaining uniform quality.

CONCRETE SUBWAY WORK.

From the article below, descriptive of the construction of the Fourth Avenue Subway, New York, it will be seen what very excellent results can be obtained by efficient methods of working, and particulars of these methods should be studied by all contractors and others engaged in work of a similar character.—ED.

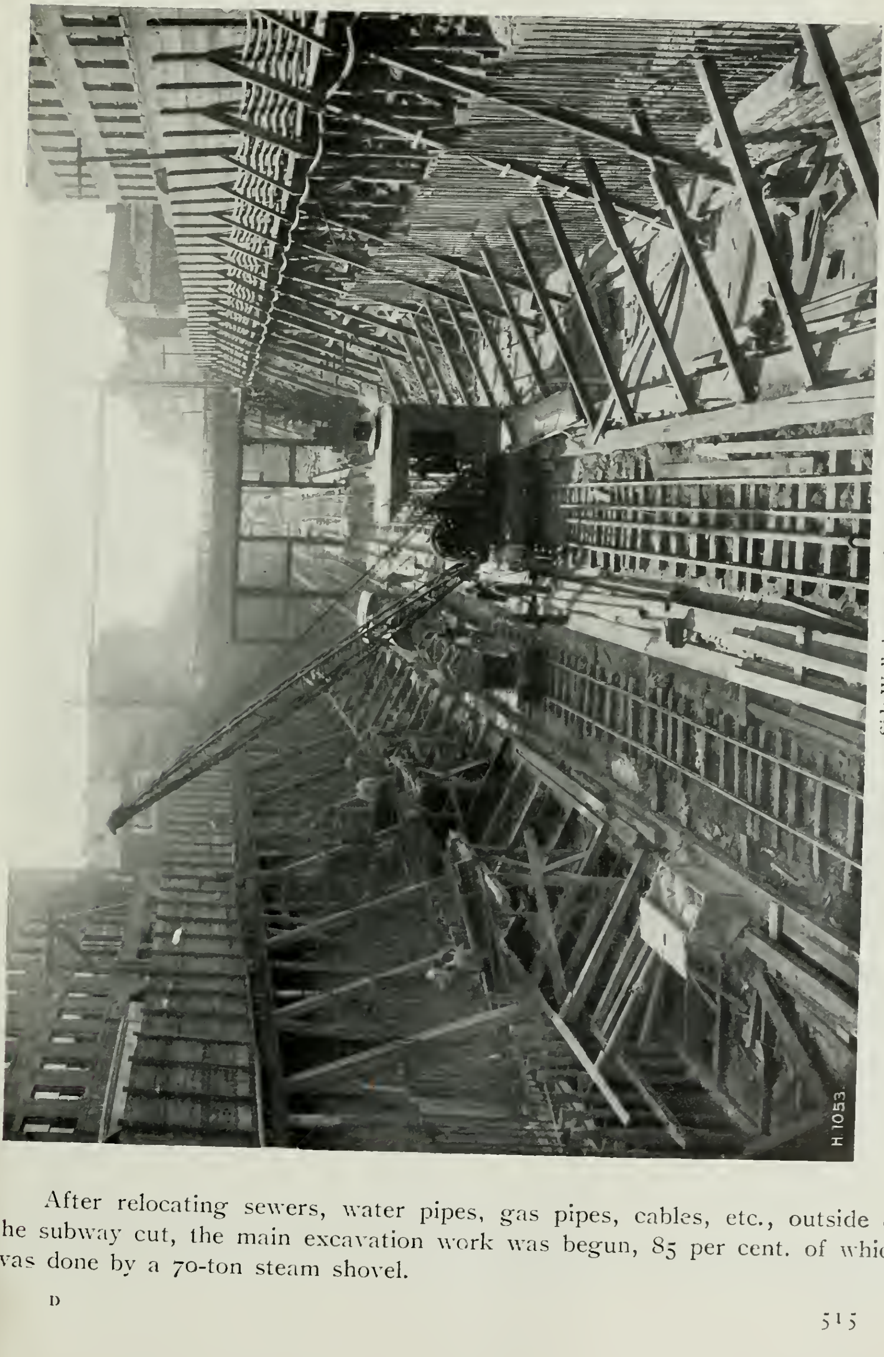
An excellent example of what remarkable results can be obtained by adopting the proper methods of excavation and construction for a given section of subway work is afforded by the work of the Tide-Water Building Co. and Mr. T. B. Bryson, of New York, in the execution of their contract for section 11-A-3 of the Fourth Avenue Subway, in Brooklyn. This is about 4,800 ft. long and extends along the hillside between Prospect Park and the Gowanus Canal. The standard section of reinforced concrete is 60 ft. wide by 19 ft. high, with a centre and two quarter walls separating each of the two local and two express tracks, which will be from 20 ft. to 30 ft. below the street pavement. The design of the two stations in this division is of structural steel, in connection with the reinforced concrete. The wall footings and all the track floors are



Interior of one of the Tubes.

Concrete Construction of the Fourth Avenue Subway, New York.

of concrete, as are also the benches located on either side of the subway, each of which contains ten four-way ducts. Hollow tile is used on the outside of the outer walls for waterproofing purposes.

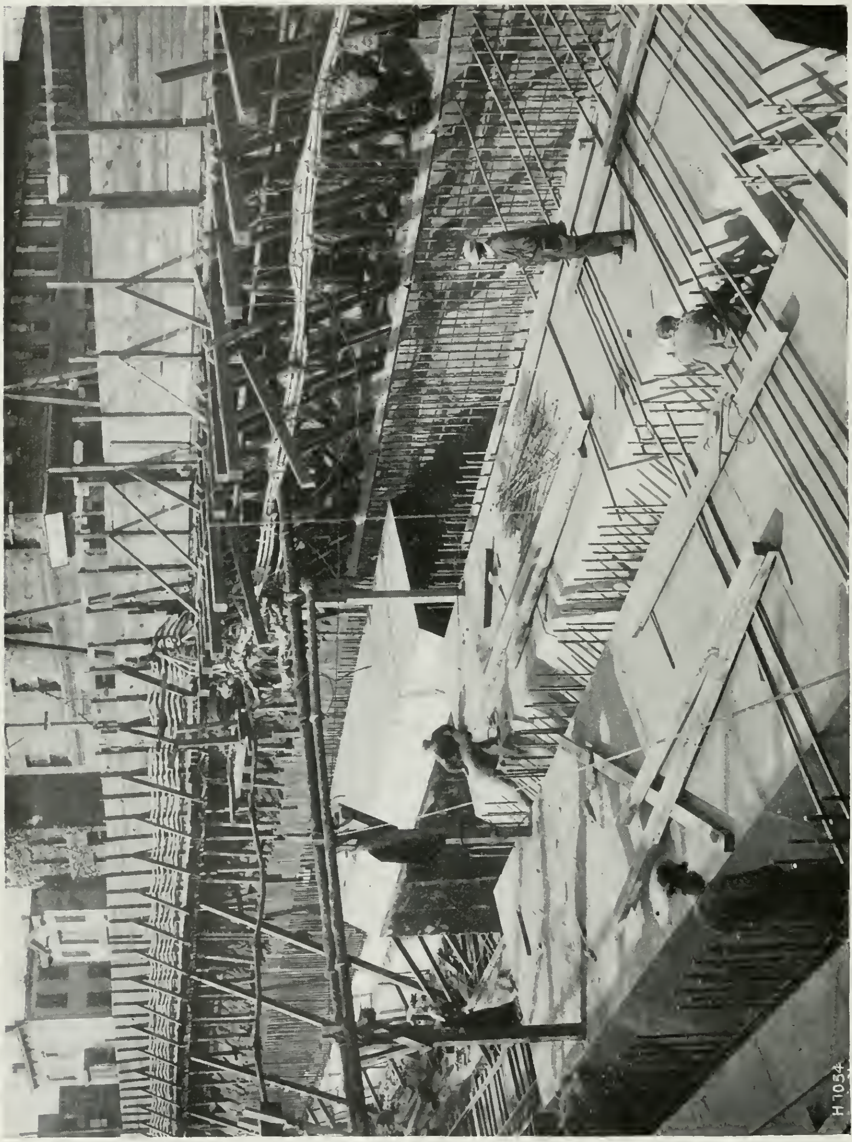


Side Walls in course of construction.

Concrete Construction of the Fourth Avenue Subway, New York.

After relocating sewers, water pipes, gas pipes, cables, etc., outside of the subway cut, the main excavation work was begun, 85 per cent. of which was done by a 70-ton steam shovel.

As it was necessary to maintain one-half of the thoroughfare for traffic at all times and also provide access to all property a temporary wooden roadway 12½ ft. wide was built on either side, with its supports outside the lines of the subway. Thus, as the material through which the excavation was made was firm, no cross-bracing was required and the cut was free from obstructions. Four cuts were made, and at such a speed that the amount the Public Service

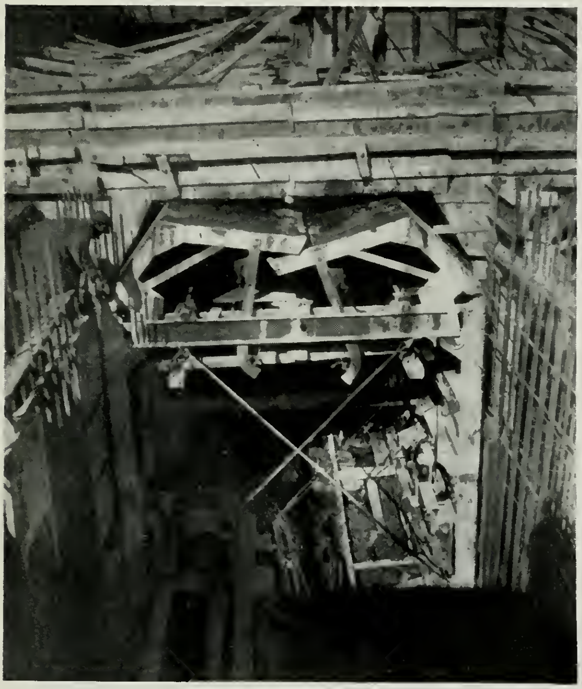


Placing Reinforcing Rods over the Forms.
CONCRETE CONSTRUCTION OF THE FOURTH AVENUE SUBWAY, NEW YORK.

Commission's engineers anticipated would be excavated in six months, from previous subway performance, was completed in two. The total amount of excavations will be in the neighbourhood of 300,000 yards place measurements.

The main concrete work consists of building the centre wall, quarter walls, outside walls (above the bench walls) and the roof. This is all done in one operation by the use of large travelling steel forms patented and built by the Blaw Collapsible Steel Centering Co., of Pittsburg.

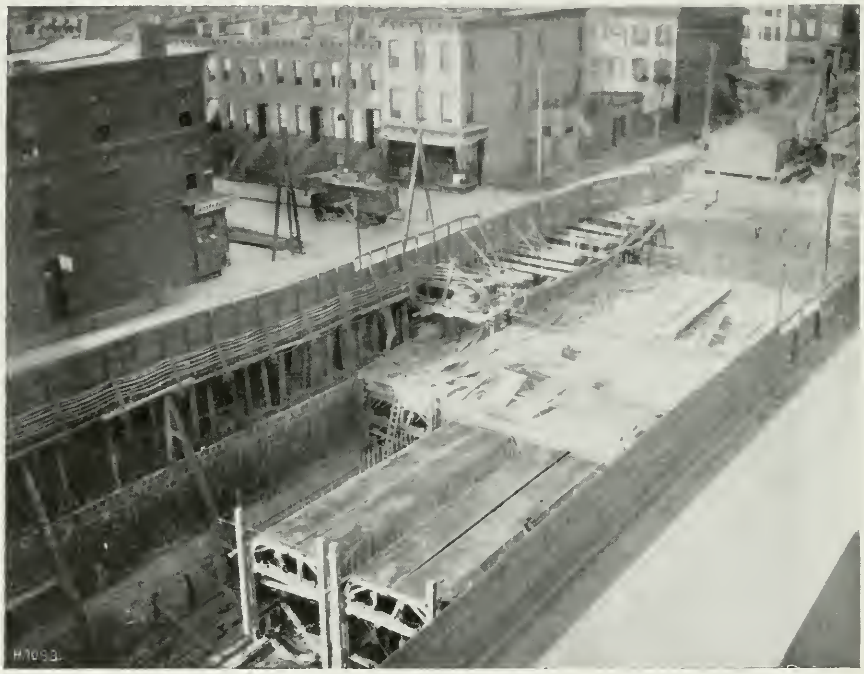
After a section of concrete has set, the spacing bolts of the forms are taken out, the forms slacked (see accompanying illustration), and crossed truss-rods with turnbuckles put in and tightened up diagonally across the tubes to hold the forms from rocking while being moved. A locomotive then draws them ahead, one tube at a time. The wall reinforcing steel and cross passage forms are all set in advance, so that when the steel forms are moved ahead the remaining reinforcement consists of roof rods or beams only. With this system the time lost in moving and setting forms is reduced to a minimum.



Collapsing a steel form to draw out of finished work.
Concrete Construction of the Fourth Avenue Subway, New York.

Of especial interest are the mixing plant and the steel forms, the arrangement of the former is well worth the careful study of concrete contractors. The plant has been so designed that with a Ransome 1-yard mixer and the conveying machinery it is capable of meeting the exacting requirements offered on days when a continuous hard 16-hour run is necessary. Its flexibility is well illustrated by the fact that, though 60 yards per hour is the average rate, 305 yards of concrete were placed in 5½ hours, and this time includes all delays, such as shifting of chutes, transportation, hoisting and dumping of buckets, etc.

With the exception of emptying the cement bags into the measuring hopper, the concrete and its ingredients are hauled mechanically from the time the materials are received at the dock until the mixed concrete is placed in the forms. The gravel and sand received at the company's wharf at the foot of Nineteenth Street and Gowanus Canal are unloaded into bins by a locomotive crane and clam shell bucket, from whence they discharge into dump cars and are hauled by 18-ton locomotives to the mixing plant in the cut at Nineteenth Street and Fourth Avenue. Coming into the section at street grade the gravel and sand are dumped into bins of about 2,000- and 1,000-yards capacity on either side of the mixing plant building. Below these bins are two separate horizontal belt conveyers discharging into a belt and bucket elevator, which carries the materials to the top of the building, where it is dumped into compartments from which lead chutes with grates to the measuring hopper. All of the conveying machinery is driven by one induction motor. The top of this measuring hopper is level with the floor, the elevation of which is the same as the tracks on which the gravel, sand and cement cars come in from the dock. The cement as received from the cars is piled around this room and the sacks are emptied into the hopper as required.



Two Sections of Forms in position.
CONCRETE CONSTRUCTION OF THE FOURTH AVENUE SUBWAY, NEW YORK.

The contents of the hopper, generally proportioned 1 : 2½ : 4½, are emptied into the 1-yard Ransome mixer located on the lower floor. The tilting chute and swinging gate of the hopper are operated by one man, who, at the same time, watches the filling of the buckets below as they come in on the cars running on parallel tracks on either side of the mixer.

The concrete, now thoroughly mixed, is hauled by locomotives to the section being constructed. Here automatic-bottom dumping buckets are raised by a locomotive crane and discharged into an elevated hopper located on and about

10 ft. above the top of the forms. The mixture then runs down an inclined chute by gravity to the point desired. As the chute is pivoted to the hopper and has a wheel at its extended end it may be shifted to all parts readily.

The collapsible travelling steel forms are made in 5-ft. sections of the shape of each tube and weigh about 8,000 lb. per 5-ft. section of single tube. At the present these are bolted together to form the standard 40-ft. sections. As the interior panels of all four tubes may be locked together by bolts no bracing was required except for the outer walls, and therefore the two express tubes are always clear.

The forms are mounted on wheels which run on rails riveted to a heavy angle which extends along and fits over the edge of the wall and bench footings. The design of a 40-ft. section of tube forms is such that the roof panels may be slacked off by the operation of four chain hoists. As this section takes place the side panels are also pulled in, leaving a clearance so that the tubes may be readily moved ahead.

In all cases the concrete has been inspected by the Public Service Commission's engineers. It is anticipated that the ultimate amount of concrete placed will be 60,000 yards, with 2,500 tons of reinforcing steel.

The Fourth Avenue Rapid Transit Subway is being built for the City of New York, under the Public Service Commission, by Mr. Thomas B. Bryson, of New York, and the Tide-Water Building Company, whose chief engineer for this work is Mr. Stephen Pearson Brown.

T-BEAM DESIGN.

By JOHN A. DAVENPORT, M.Sc., A.M.Inst.C.E.

(Author of "Graphical Reinforced Concrete Design.")

In our last issue we published an article on the design of T-beams in reinforced concrete, but as the subject is one of special interest we give below another dealing with the same matter. — ED.

IN designing reinforced concrete T-beams the following data is usually known :—

(1) The total load (useful and dead load) per sq. ft. of floor slab ;

(2) The distance apart of the T-beams ;

(3) The span of the T-beams ;

(4) The thickness of the floor slab which forms the top flange of the beams, and the design must be completed in accordance with the R.I.B.A. recommendations.

The load per ft. run of the beam will be given by the product (1) × (2), which we may call w ; then if we call the span of the beam l, the moments of the external forces or bending moments are as follows (maximum values being only considered) :—

For a continuous beam

$$B=\frac{wl^2}{12}\times 12=wl^2 \text{ lb. in.} \tag{1}$$

and for a non-continuous beam

$$B=\frac{wl^2}{8}\times 12=1{\cdot}5wl^2 \text{ lb. in.} \tag{2}$$

w being in lbs. per ft., and l in ft.

Now the design must be such that the moment of the internal forces, or moment of resistance, at any section shall not be less than the bending moment applied there ; and, in order that it may be economical, the moment of resistance must equal the bending moment.

The moment of resistance is given in terms of the dimensions of the section in the following formulæ, which occur in the 2nd Report of the R.I.B.A. Joint Committee on Reinforced Concrete :—

$$R_c=cbdd_s\frac{(s_1^3+4mrs_1^2-12mrs_1+12mr)}{6(s_1^2+2mr)} \tag{3}$$

$$R_t=tbd^2\frac{(s^3+4mrs_1^2-12mrs_1+12mr)}{6m(2-s_1)} \tag{4}$$

R_c being the moment of resistance of the concrete, and R_t the moment of resistance of the steel. It may be noted that if the beam be correctly and economically designed, expressions (3) and (4) will be identical ; there are, however, often practical reasons for a slight difference between the two.

The distance between the centres of compression and tension is called the "lever arm," and is given in the Report by:

$$a = d\left[1 - \frac{s_1}{3}\left(\frac{3n_1 - 2s_1}{2n_1 - s_1}\right)\right] \tag{5}$$

Now in designing a T-beam, the following quantities are required for the plans (see *Fig.* 1):

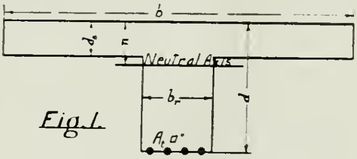


Fig. 1.

d_s, the thickness of the floor slab;

d, the total effective depth of beam (by combining d with d_s and the cover, we can get the total depth of beam below the slab);

b_r, the breadth of the rib or web;

A_t, the area of the main reinforcement.

Of these quantities d_s is usually known, as the slab is calculated before the beam; and there remain d, b_r, and A_t to be calculated. Unfortunately neither equations (3) and (4), nor any other equations in the Report, enable us to calculate b_r, but there is a rule which states that b_r should not be less than one-sixth b, b being the width of top flange, which may be taken as acting with the web. There are three additional statements for determining b in terms of d_s, the distance between the beams and the span of the beams.

In the usual method of designing T-beams the value of b_r is chosen as a matter of experience, and having made this choice the quantities d and A_t are still required.

To find these values by using equations (3) and (4), or any others given in the Report, is a most difficult process of trial and error, even when c and t be given particular values; and it is now proposed to deduce approximate equations which will enable T-beams to be designed more easily.

Generally speaking, the most economical design of T-beam will be that in which the steel and concrete are stressed up to their limiting values; although in the case of a very thin and deep web this may not be true, owing to the relatively high cost of the centering. But in the usual design there is little chance of such a case occurring. On this account, then, the working stresses are taken as $t = 16{,}000$ lbs. per sq. in., and $c = 600$ lbs. per sq. in., in the following investigation. Further, T-beams with the neutral axis in the web will only be considered, the case of T-beams with the neutral axis in the slab being included in plain beams.

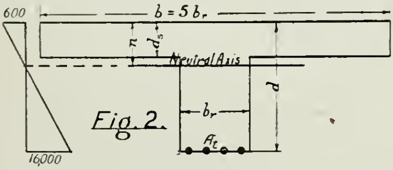


Fig. 2.

It is assumed that $b = 5b_r$ (the rule in the Report allows the limiting value $b = 6b_r$, subject to certain conditions), and that the small compressive resistance in the top of the web is negligible.

The former assumption much simplifies the expressions and is sufficiently accurate for practical purposes, as will be seen. Referring now to *Fig.* 2, standard symbols being used,

$$\frac{n}{d-n} = \frac{mc}{t} = \frac{15 \times 600}{16{,}000} = \frac{9}{16} \tag{6}$$

rom which we get $n = \frac{9}{25}d$, and $(d-n) = \frac{16}{25}d$ (7)

The compressive stress at the underside of the flange or slab is given by:

$$\frac{600}{n}=\frac{C_u}{n-d_s}, \text{ whence}$$

$$C_u=600\left(1-\frac{d_s}{n}\right)=600\left(1-\frac{25d_s}{9d}\right) \qquad (8)$$

from (7). The mean stress over the slab is:

$$C_{\text{mean}}=\frac{600+C_u}{2}=300\left(2-\frac{25d_s}{9d}\right) \qquad (9)$$

from (8). The total compressive resistance of the flange will be given by multiplying expression (9) by $b\times d_s$, the effective area. Thus:

$$\text{Total compression}=300bd_s\left(2-\frac{25d_s}{9d}\right) \qquad (10)$$

The moment of resistance of the section, so far as the compression is concerned, is given by the product of (10) and the lever arm, which we must now determine.

Again, an assumption will be made which, while being only slightly inaccurate, greatly simplifies the equations.

When the neutral axis is in the underside of the slab the centre of compression is at a distance $0{\cdot}333d_s$ from the extreme fibre in compression, while at the other extreme—*i.e.*, a beam in which the slab is so thin, compared with the total depth, that the compressive stress is practically uniform across it—the centre of compression is at a distance $0{\cdot}5d_s$ from the extreme fibre. The mean of these two distances is $0{\cdot}42d_s$, and this is taken as the distance of the centre of compression from the extreme fibre. The lever arm then becomes:

$$a=d-0{\cdot}42d_s \qquad (\text{A})$$

and taking the product (10) × (A), we get the moment of resistance

$$R_c=300bd_s\left[2d-3{\cdot}62d_s+\frac{1{\cdot}17d_s^2}{d}\right] \qquad (11)$$

and putting in the value $b=5b_r$, it becomes:

$$R_c=1{,}500\,b_rd_s\left[2d-3{\cdot}62d_s+\frac{1{\cdot}17d_s^2}{d}\right] \qquad (\text{B})$$

The moment of resistance of the section, so far as the tension is concerned, is given by multiplying the total tension by the lever arm, so that:

$$R_t=16{,}000A_t(d-0{\cdot}42d_s) \qquad (\text{C})$$

A simpler form of expression (B) is given in the following, for use only when the method of trial and error cannot be avoided; but the results obtained by it should be checked by using the expression (B).

$$R_c=1{,}500b_rd_s[2d-3{\cdot}333d_s] \qquad (\text{D})$$

As already explained, R_c and R_t should not be less than the applied bending moment B; and for economy, they should both be equal to it, so that from this fact we get, corresponding to equations (B), (C) and (D), the expressions:

$$B=1{,}500b_rd_s\left[2d-3{\cdot}62d_s+\frac{1{\cdot}17d_s^2}{d}\right] \qquad (\text{B}_1)$$

$$B=16{,}000A_t[d-0{\cdot}42d_s] \qquad (\text{C}_1)$$

$$B=1{,}500b_rd_s[2d-3{\cdot}333d_s] \qquad (\text{D}_1)$$

To design a T-beam with these equations proceed as follows:—

First find, from the total slab load and the distance apart of the beams, the load per foot run; then using this quantity, together with the span, in equation (1) or (2), calculate the bending moment. Next choose a suitable value for b_r, a simple matter to the experienced designer, and putting this in (B_1), calculate d. Use this value of d in equation (C_1) and calculate A_t. Calculate the depth of beam below the slab by adding the cover to d, and subtracting d_s from the sum. Thus the important quantities—

Breadth of web,
Depth of web below the slab,
Area of main reinforcement,

are known and can be stated on the plans. Check the design by applying equations (3) and (4). In the event of a difficulty arising regarding the choice of b_r, determine a suitable value by the method of trial and error, using (D_1).

NOTE.—Before the design is complete the shear force should be dealt with, but as it is not intended to deal with shear force in this article its consideration is left out.

EXAMPLE.

A floor slab carrying a useful load of $1\frac{1}{4}$ cwt. per sq. ft. is supported by beams 9 ft. apart, the span of the beams being 20 ft. The slab is already calculated to have a total thickness of $4\frac{1}{2}$ in.; the beams are freely supported at the ends and have 2 in. of cover protecting the reinforcement. Design a suitable section.

Useful load = 140 lb. per square foot.

Dead load $= \dfrac{4\frac{1}{2}\times 150}{12}$ + beam load $= 56+4=60$ lb. per sq. ft.

Total slab load = 200 lb. per sq. ft.

Load per ft. run of beam $w=200\times 9=1{,}800$ lb.

Span of beams = 20 ft.

$B=1{\cdot}5wl^2=1{\cdot}5\times 1{,}800\times 400=1{,}080{,}000$ lb. in.

Take $\boldsymbol{b}_r=9$ in., then using equation (B_1) we get:

$$1{,}080{,}000=1{,}500\times 9\times 4\tfrac{1}{2}\left[2d-3{\cdot}62\times 4{\cdot}5+\frac{1{\cdot}17\times 4{\cdot}5\times 4{\cdot}5}{d}\right]\text{, or}$$

$$17{\cdot}8=2d-16{\cdot}3+\frac{23{\cdot}7}{d}$$

from which we get the quadratic:

$$d^2-17{\cdot}05d+11{\cdot}85=0$$

$$d=\frac{17{\cdot}05\pm\sqrt{17{\cdot}05^2-4\times 11{\cdot}85}}{2}$$

$$d=\frac{17{\cdot}05\pm 15{\cdot}6}{2}=16{\cdot}32\text{, say 16 in.}$$

Now to find the lever arm,

$$a=16-0{\cdot}42\times 4{\cdot}5=14{\cdot}11\text{ in.,}$$

and the area of the reinforcement is given by

$$A_t=\frac{1{,}080{,}000}{16{,}000\times 14{\cdot}11}=4{\cdot}78\text{ sq. in.}$$

This area may be obtained by taking 5 bars 1¼ in. dia. (total area 4·97 sq. in.), which is slightly in excess of the calculated amount. Adding now the 2-in. cover to *d*, and subtracting the thickness of the slab, we get the depth of beam below the slab, equals 13½ in., and the section is shown in *Fig.* 3—*i.e.*, a beam 13½ in. × 9 in., with 5 bars 1¼ in. dia.

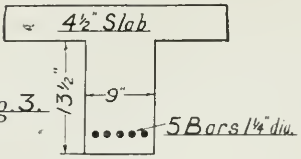


Fig. 3.

To check by using equations (3) and (4), we have the following quantities:

$c = 600$

$t = 16{,}000$

$b = 5 \times 9 = 45$ in.

$d_s = 4{\cdot}5$ in.

$d = 16$ in.

$s_1 = \frac{4{\cdot}5}{16} = 0{\cdot}281,\ s_1^2 = 0{\cdot}079,\ s_1^3 = 0{\cdot}022$

$m = 15$

$r = \frac{4{\cdot}97}{45 \times 16} = 0{\cdot}0069$, then

$$R_c = 600 \times 45 \times 16 \times 4\tfrac{1}{2} \frac{[0{\cdot}022 + (4 \times 15 \times 0{\cdot}0069 \times 0{\cdot}079) - (12 \times 15 \times 0{\cdot}0069 \times 0{\cdot}281) + (12 \times 15 \times 0{\cdot}0069)]}{6\,[0{\cdot}079 + (2 \times 15 \times 0{\cdot}0069)]}$$

$$= 1{,}944{,}000 \left[\frac{0{\cdot}9457}{1{\cdot}716}\right]$$

$$= 1{,}072{,}000 \text{ lb. in.}$$

$$R_t = 16{,}000 \times 45 \times 16 \times 16 \left[\frac{0{\cdot}9457}{6 \times 15(2 - 0{\cdot}281)}\right]$$

$$= 184{,}320{,}000 \left[\frac{0{\cdot}9457}{154{\cdot}7}\right]$$

$$= 1{,}126{,}000 \text{ lb. in.}$$

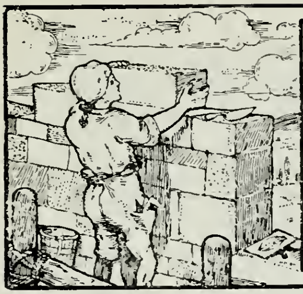
The value of R_c is 0·8% low, because *d* was taken 16 in., instead of 16·32 as calculated. For practical reasons *d* must either be 16 or 16½ in., and if the latter value be taken it will be found that R_c, above, is larger than the bending moment. As R_c, with $d = 16$, is only 0·8% low, this value will probably satisfy the authorities; if not, then *d* must be made 16½ in. The value of R_t is higher than the bending moment, because we are compelled to take an area slightly in excess of the calculated value. The value of *b* in this example is smaller than that permitted by the rules of the Report, which rules allow it to have the smallest of the following values:

$$b = \frac{l}{3} = 6{\cdot}66 \text{ ft.}$$

$$b = \tfrac{3}{4} \text{ slab span} = 6{\cdot}75 \text{ ft.}$$

$$b = 15 d_s = 5{\cdot}62 \text{ ft.} = 67 \text{ in.}$$

We might, therefore, if we wished, take $b = 67$ in., instead of 45 in., but then b_r should not be less than $\frac{67}{6} = 11$ in., and a comparatively shallow and uneconomical section would result.

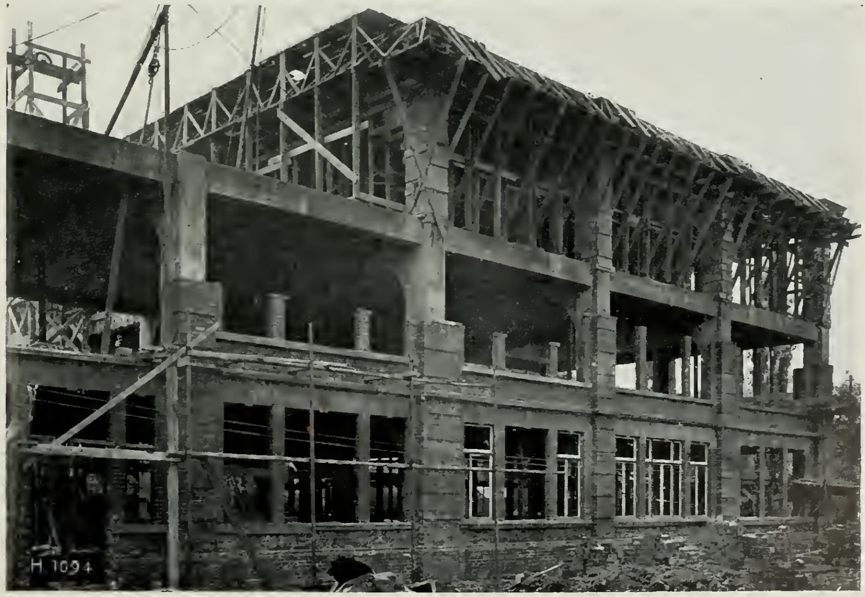


REINFORCED CONCRETE BUILDINGS AT ROWNTREE'S COCOA WORKS.

Below we give particulars of numerous buildings which have been erected at Rowntree's Cocoa Works, York, on a system of reinforced concrete designed and patented by Messrs. Rowntree's architect, which should be of interest to our readers.—ED.

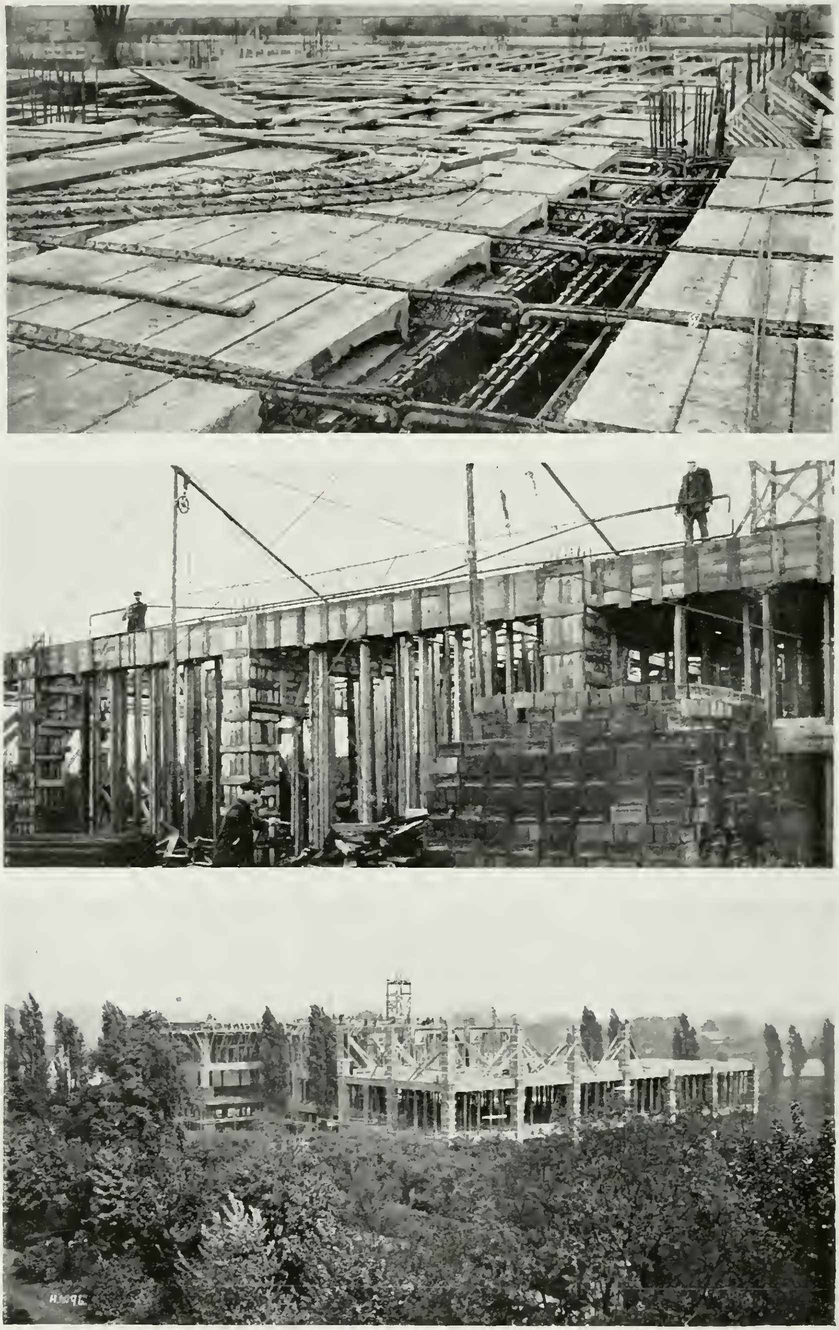
THE new dining-room block which is now in course of erection for Messrs. Rowntree's, of York, contains about 8,000 square yards of flooring, and when completed will be one of the finest buildings of its class in the country. It is capable of dining at one time 3,000 persons, and will be equipped with the latest and most up-to-date kitchens, which are designed to come on the top floor, from which several electrically-driven hoists will serve each floor. The building also contains spacious rooms for boys' scholastic classes, wood-working shops, etc., as well as social club rooms for men and boys.

A large lecture hall is also provided, and there is ample provision for girls' domestic classes, which when fitted up will compare with the largest and most advanced type of secondary schools in the country. The south wing, which is



New Dining-room Block in course of construction.
REINFORCED CONCRETE BUILDINGS AT ROWNTREE'S COCOA WORKS.

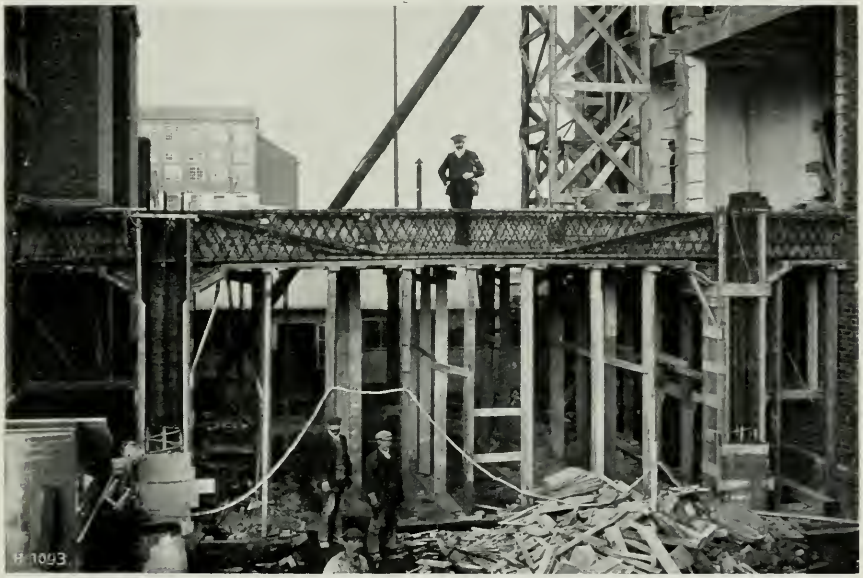
150 ft. by 50 ft., is devoted to the gymnasia for boys and girls. A handsome verandah, 15 ft. wide, runs the full length of the south elevation. The building



Three stages in the construction of Dining-room Block.
Reinforced Concrete Buildings at Rowntree's Cocoa Works.

will be be heated throughout by atmospheric pressure steam, and lighted by electricity.

The feature of the building from a constructional point of view is the huge spans obtained by the use of Mr. W. J. Swain's patented systems of reinforced concrete and reinforced concrete hollow block floors. The clear spans of bays are 46 ft. by 26 ft. without columns or visible beams, the thickness of the hollow floors being only 9 in. Many thousands of pounds' worth of work have been executed on this cross-braced system, which the architect claims to be one of the most scientifically constructed methods yet invented for reinforced concrete, especially for large spans and heavy loads. Each truss forms a lattice girder, and these trusses are put together on the ground and hoisted into position as though they were R. S. J.'s. Each truss can be examined on the ground and passed before hoisting, as it cannot become deranged when once completed, and it is impossible for the workmen to shift the position of the steel rods or lattice members during the filling or punning of the concrete.



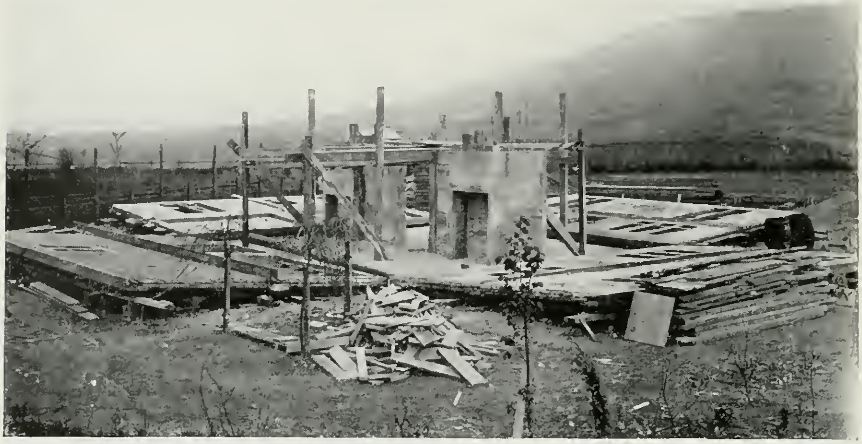
View showing trusses in beam box for three deck bridge between two factories.
REINFORCED CONCRETE BUILDINGS AT ROWNTREE'S COCOA WORKS.

The lattice bracing also provides for varied stresses due to eccentric loading or settlements, which no other system has yet achieved.

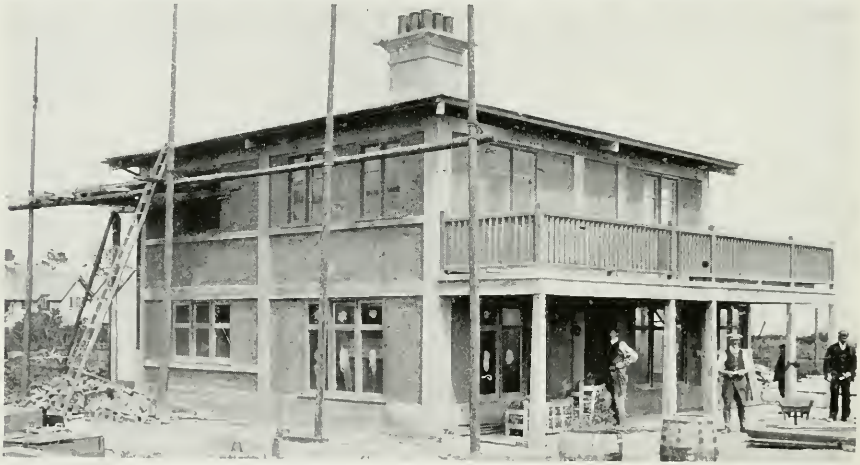
The official tests of floors made in this system have shown excellent results —on one floor the clear span of which was 18 ft. 6 in. a test of 120 tons, being $7\frac{1}{2}$ cwt. per sq. ft., was employed, and the deflection obtained was only $\frac{3}{64}$ in. and no permanent set. For big span bridges this system is particularly suitable. It is impossible for columns to buckle when reinforced on the four sides with this system of cross-bracing, the trusses of which are bound together at the angles. One of the great advantages claimed is that ordinary round commercial steel bars are employed, and the whole is put together, with ease, at small cost, by unskilled labour; and the cross-bracing allows of any number of bars being employed in the trusses. These trusses could be made in any town and shipped to another without fear of dislocation of any of the members.

Several of these trusses have been made over 60 ft. in length and hoisted into position.

Illustrations are also shown on this page of a novel house erected last year on the same system, in which the walls were cast upon the ground with doors and windows complete, and hoisted into position. From start to finish, including decorating, furnishing, and occupation, was only twelve weeks.



Reinforced Concrete Walls on ground, with windows and doors complete, ready for hoisting into position.

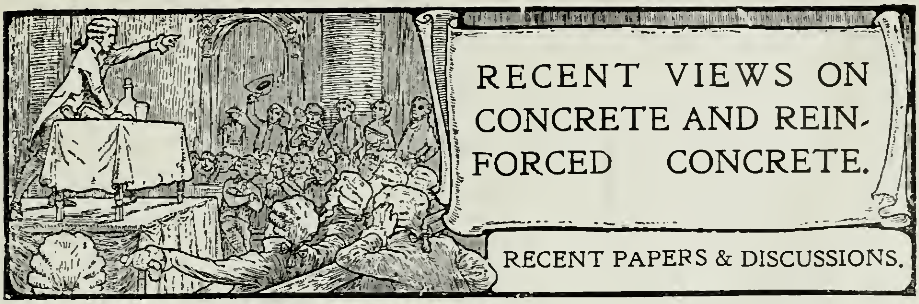


Concrete House completed.

REINFORCED CONCRETE BUILDINGS AT ROWNTREE'S COCOA WORKS.

Mr. W. J. Swain is the resident architect for Messrs. Rowntree's, York, and carries out the whole of the designing and calculations himself, the work being done by the firm's own workmen. They keep a staff of nearly 200 in the building department alone.

The factories cover about twelve acres of ground-floor space, though many of the buildings are six and seven stories high.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE CONCRETE INSTITUTE.

THE TRUE BENDING MOMENTS OF BEAMS WITH VARIOUS DEGREES OF FIXITY.

By MAURICE BÉHAR,
Civil Engineer, Ecole des Ponts et Chaussées.

THE object of this paper was to draw the attention of all those concerned with reinforced concrete to the disadvantages which would result from the application of certain theories and formulæ, concerning which there had recently been a certain amount of controversy; also to show, by means of examples, the excessive increase in the weight of the steel bars in beams and posts in reinforced concrete, due to the application of these methods.

Beams.—Until now almost all the engineers specialising in reinforced concrete calculate beams by applying the usual laws of mechanics, or, in other words, the laws governing the strength of materials. Nevertheless, for the main beams and secondary beams fixed at their extremities to other members in reinforced concrete, such as walls, posts, or other beams, the specialist engineers usually admit that a certain amount of fixity takes place at these points of junction, and concerning this it is usual to make certain assumptions.

Some engineers consider that the fixation is partial on the support, and has for its value $\frac{WL}{24}$. In this case they calculate their moment at the middle of the beam, sufficiently strong to resist a bending moment of $\frac{WL}{12}$, which is the corresponding moment to the one stated above.

Other engineers, and we think they are in the majority, consider that the bending moment should be $\frac{WL}{10}$ in the middle of the beam, and in this case they provide at the points of support a section of steel capable of resisting a corresponding moment of $\frac{WL}{40}$.

Again, other specialists consider that reinforced concrete beams extending over several spans must be considered as continuous, and they apply to the calculation of these beams the usual rules governing continuity.

In addition to the three above-mentioned methods of calculating a beam in reinforced concrete fixed upon its two points of support, it is now my intention to consider the method which has been put forward for official sanction, and which would have the effect of stipulating a bending moment of $\frac{WL}{12}$ in the middle of the beam and $-\frac{WL}{12}$ at the points of support.

Of these four methods of calculation, let us consider the question of which one produces the maximum of safety in the construction.

1. Let us assume that the bending moment in the middle is $\frac{WL}{12}$, and that the corresponding moment at the points of support is $\frac{WL}{24}$. It may happen that, on account of the continuity of two consecutive beams and the distribution of the loads on the spans, the moment at the point of support becomes greater than the one provided for. In this case fissures will occur at this point.

Let us now assume by exaggeration that these fissures render the points of support absolutely free. The middle of the beam will then support a bending moment equal to $\frac{WL}{8}$, but as the beam has been calculated for a bending moment in the middle of $\frac{WL}{12}$, the factor of safety will become

$$4\times\frac{8}{12}=2{\cdot}666 \text{ instead of } 4.$$

The beam will not collapse on this account, inasmuch as the concrete in the lower part of the beam situated underneath the neutral axis is working in extension, which has not been taken into account in the calculations.

2. Let us now assume that the bending moment in the middle is $\frac{WL}{10}$ and the corresponding moment at the supports is equal to $\frac{WL}{40}$. The method of reasoning is the same as for the above, except that in the case of the transformation of the point of support into a free support the factor of safety becomes

$$4\times\frac{8}{10}=3{\cdot}2 \text{ instead of } 4.$$

3. Let us now consider the continuous beams. In this case the moments at the supports are generally greater than $\frac{WL}{12}$.

Concerning the moments in the middle of the spans, they will often be much less than $\frac{WL}{12}$. We must remember, however, that although in structural steelwork it is possible to obtain perfect continuity between two consecutive spans, it is not so with reinforced concrete, where the continuity depends upon the adherence of the concrete to the bars provided in the upper portion of the beam at the points of support.

I am of opinion that the method of constructing consecutive beams in reinforced concrete does not ensure such perfect continuity as that which may be obtained in consecutive metallic girders. If owing to this lack of solidarity between two consecutive spans, or if owing to bad construction the point of support should give way and become transformed into a free support, then the middle of the beam, which may in certain cases have been calculated with a moment less than $\frac{WL}{12}$, will have a factor of safety less than 2·66, and in certain cases even less than 2. It is obvious that the construction in this case becomes endangered, and the danger is all the greater because if one of the beams gives way the others will give way in turn, on account of the fact that the continuity upon which we relied will have disappeared owing to the failure of the first element.

It is for this reason that in steel construction, when there are several continuous spans, it is usual to divide these into a series of portion of three or four consecutive continuous spans, each portion being separated by free supports.

4. If we consider $\frac{WL}{12}$ in the middle and $-\frac{WL}{12}$ at the points of support, this case certainly has the effect of producing a greater safety than those above mentioned. In the middle there is the same drawback as in the first case, but not the one due to continuity. At the supports, however, this case is weaker than the case of continuity,

although it has very nearly the same value. On the other hand, the latter method obliges the engineer to provide, as in the case of continuity, in the bottom compressed portion of the beam, and at the points of support, a considerable section of steel, and this section will be all the greater at these two points, on account of the fact that, if this principle is admitted in the Official Regulations, the section of steel required will have to be calculated by applying a stress in the steel equal to fifteen times the stress of the concrete, taken at the axis of the reinforcement employed.

Now, as a matter of fact, a large number of examples of principal beams and secondary beams which I have calculated with $\frac{WL}{12}$ at the supports, have proved to me that the working stress of the steel in compression is always below 8,000 lb. per sq. in., and that in secondary beams this stress rarely attains 6,000 lb. per sq. in. The result is that one is often obliged to provide such a considerable number of steel bars, that it is materially impossible to place them in the small compressed area of the concrete, in the bottom of the beam.

A close examination of the four methods of calculation mentioned above has led me to conclude that, as far as stability is concerned, the moment of $\frac{WL}{12}$ in the middle and at the supports gives a greater security. $-\frac{WL}{24}$ or $-\frac{WL}{40}$ at the supports and their corresponding values of $\frac{WL}{12}$ or $\frac{WL}{10}$ in the middle would never actually cause the collapse of a beam. The most that could happen in such a case would be, that the floor slab over the beams would show fissures, whereas, if we consider the case of continuity, we find that this may prove dangerous if the workmanship is bad or the materials unsuitable.

From an economical point of view, the semi-fixation with moments of $-\frac{WL}{24}$ at the support and $\frac{WL}{12}$ at the middle, requires about the same amount of steel as the partial fixation with moments of $-\frac{WL}{40}$ at the supports and $\frac{WL}{10}$ at the middle.

Concerning $-\frac{WL}{12}$ at the supports and $\frac{WL}{12}$ in the middle, when it is possible to actually construct the beam by accommodating the amount of steel required, I have found that these values bring about an increase 40 to 50 per cent. greater than required in the cases of semi or partial fixation mentioned above, and that in certain cases the excess of steel may be as much as 100 per cent.

Concerning the case of continuity; if we are obliged to provide in the lower portion of the beams the necessary section of steel to take up the compression at the points of support, and if we put aside the method which consists of reinforcing this portion by means of spirals, the weight of the bars would be less than that due to $\frac{WL}{12}$ at the supports and in the middle, but the weight of steel would be superior to that produced by the formula of semi or partial fixation.

I understand that for various reasons the method of reinforcing the concrete in the beams by means of spirals would not be considered or authorised in the new Regulations, and it is doubtful whether this particular method could meet the difficulty in an economical manner. If, however, the new Regulations were to allow the use of spirals, to increase the compression of beams, and if it were found that this was the only possible way of solving the difficulty of the compression, then it is obvious that the Regulations would have the effect of favouring, to the exclusion of all others, one particular method only for which a patent has been obtained, thereby creating a monopoly.

It is therefore absolutely necessary that the proposed Official Regulations should not have the effect of obliging engineers to calculate beams fixed at both extremities with a bending moment of $-\frac{WL}{12}$ at the supports and $\frac{WL}{12}$ at the middle, and I sincerely trust that the authorities, who are at present drafting out new Regulations, will simply stipulate the application of the usual laws of mechanics for the determination of the dimensions of beams in reinforced concrete; and also hope that the case of semi-fixed beams, which is in no ways empirical, and which is absolutely in conformity with the laws of mechanics, shall be considered and shall find its proper place in the new Rules.

Compression in the Upper Portion of the Beams and in the Middle of the Span.—I understand that it is intended to introduce in the new Rules a clause stipulating that the width *b* of the slab working in compression together with the beam shall be equal to the smallest of the following values:—

(*a*) To one-third of the effective span of the beam.
(*b*) Or three-quarters of the distance centre to centre between beams.
(*c*) Or six times the thickness of the beam.
(*d*) Or fifteen times the thickness of the slab.

I have found that the cases of *c* or *d* are those which will be usually applied, especially for principal beams. It is obvious that owing to this the compression will almost in every case be insufficient at the centre of these beams. As, on the other hand, the upper bars are usually placed 2 in. from the upper surface of the beam, the working stress of the steel will rarely reach 8,000 lb. per sq. in. We shall therefore be obliged to provide a relatively considerable section of steel in the top portion of the beam, and in certain cases this section might even exceed the section of the steel in tension, unless the Rules authorise engineers to consider the beam in the middle as being practically a steel girder, and in this case the sections at the top and at the bottom would be equal; but of course the compression of the concrete would have to be neglected.

Now, if we examine a reinforced concrete construction composed of slabs, secondary beams and principal beams (for instance, floors or retaining wall), the construction presents an absolutely monolithic and rigid table, and as the component members are working together I would suggest that in reality it would be possible to assume that the entire distance between centre to centre of the secondary beams or main beams is working in compression—in other words, that the entire area of the slab could be taken for the resistance in compression of the beams.

The only objection to this method of procedure is that to the compression of the slab itself considered separately we must add the compression produced by the secondary beams—that is, if we only consider a slab and secondary beam.

If we consider the principal beams we find that to the compression produced by the latter we must add the compression due to the secondary beam. It is possible that the resultant of these two forces may produce a compressive stress superior to 600 lb. per sq. in., especially in the case where the compression of the slab is working with the secondary beam, inasmuch as the slab itself is usually calculated for a stress of 600 lb. at the extreme compressed fibre, but when the slab is working with the principal beam it is only influenced by the compression of the secondary beam, and as in reality the entire table or slab is working in the direction of the secondary beams as well as in the direction of the principal beams, the result is that the rates of compression at the extreme compressed fibre are comparatively low, and the resultant rarely attains 600 lb. per sq. in., except in the case where the section area of the concrete being insufficient, the designer has been obliged to introduce a certain section of steel in compression.

It follows from the above method of reasoning that it would be possible to retain the clauses *a*, *b*, and *d* for the secondary beams, but that it would be advisable to suppress the clause *c*, which I consider is unnecessary as applied to secondary beams, and I am of opinion that clauses *a* and *b* only should remain for the principal beams.

It is perhaps for the reason mentioned above that the French Regulations for reinforced concrete stipulate that three-quarters only of the slab application must be taken for the compressive resistance of secondary beams, or one-third of the span of the beam, whichever is the smallest; but it is to be noticed that these Rules have not prescribed anything concerning the principal beams, so that in France engineers are free to take the entire width of the slab for the compressive resistance of the principal beams.

I wish to make it clear that I do not actually advise that this should be done, but that clauses *a* and *b* should subsist for the calculation of the compressive resistance of principal beams.

Reinforced Concrete Lintels or Beams without Slabs.—I am under the impression that the proposed Regulations have not put forward any rules to deal with this case. If, therefore, we apply to these beams the formulæ applicable to rectangular beams, we would have to take for the stress of the steel in compression fifteen times the value of the stress of the concrete at the centre of gravity of the section of the compressed bars—namely, about 8,000 lb. per sq. in. A reinforced concrete lintel will therefore have a weight of steel often superior to that of a steel girder or joist, and on this account there would be no advantage in future in adopting reinforced concrete for this kind of work.

Posts or Pillars in Reinforced Concrete.—Concerning the calculations of posts or pillars, I take it that the formula which is intended to be adopted will be—

$$P=c\ (A+(m-1)A_v),$$

in which c represents the working stress of the concrete in direct compression, A the effective area of the pillar—that is to say, the section of the pillar after deducting the area of concrete situated between the external faces of the pillar and the vertical bars, A_v being the section of the vertical bars and m equal to 15.

Although this formula may be applied to the case in which the transverse reinforcement is taken into account, and where the diameter of the transverse reinforcement and the spacing of same per foot run would lead us to take for c much higher values than 600 lb. per sq. in., I fail to see why this formula should be applied to ordinary pillars, seeing that for purely constructional reasons only transverse reinforcement has been introduced for the binding together of the principal bars during the concreting operation, and in this case it would appear to me that the total section of the concrete should be taken into account, which has been, and is, the usual practice of most of the experienced designers in reinforced concrete.

If an additional stipulation is considered necessary for further safety, I would suggest that this should take the form of reducing the rate of 600 lb. per sq. in. of the concrete in compression, in conformity with Rankine's formula.

DISCUSSION.

Professor Henry Adams, M.Inst.C.E.—He agreed with the majority of the statements made in this paper, but some of them he did not agree with, and others he did not follow.

Continuous beams were the gist of the whole paper. The author said: "In this case the moments at the supports are generally greater than $\frac{WL}{12}$," but there was only one usual case to his (Prof. Adams's) knowledge where that occurred, and that was in the end span of a beam. With regard to continuity between two consecutive spans, the author admitted that in structural steelwork it was possible to obtain perfect continuity by the construction of the girders, but in his opinion it was equally possible to obtain continuity between girders of adjacent spans in reinforced concrete. If it were a question in the one case of the shear strength of the rivets, in the other case it was the adhesion of the material to the concrete, and with proper designing there was no reason for putting the one class of work in a different category from the other.

The author also stated that "it is obvious that the construction in this case becomes endangered, and the danger is all the greater because if one of the beams gives way the others will give way in turn." That is comparing it with a railway viaduct, where if one arch fails the whole series of arches must of necessity fail up to the stop abutment, and that was why a stop abutment was put in. As in the case of continuous beams, the severance of the continuity at one point did not affect the others; it only affected the immediate contiguous beams and made them equivalent to the end beam of a series.

In the four different methods to be compared in finding the width of flange of a T-beam, the third one which the author objected to he believed is likely to be removed from the Regulations. It was one that did not compare readily with the other three, and, with the author, he hoped that it would be deleted in the final Regulations.

With regard to the beams taking into account part of the floor slab, his view was that in the majority of cases the main reinforcement was in one direction only in the slab, and in that direction it must not be taken into account as the flange of a T-beam. When there are main beams, cross-beams, and the floor slab, the main reinforcement of the floor slab would come in the direction of the main beams—that is, from one cross-beam to the other—and in that case the main beams should be taken as rectangular beams, only the cross-beams being taken as T-beams.

Mentioning pillars, the author said that "the transverse reinforcement had been introduced for purely constructional reasons, only for bonding together the principal bars during the concreting operation." He thought one of the chief reasons for the transverse reinforcement was to prevent the bars from bulging under the vertical load.

With regard to pillars, ·8 per cent. of the section of concrete for the steel was a good proportion, but it was not always possible to keep to it. There were other considerations that affected the diameter of the pillar. When you want to keep it very small you might have to put more than that percentage of steel, and if you wished to keep it the same width as the main beam, then it might be that less than that percentage of steel is required.

He would say in conclusion that in his opinion all the stresses in reinforced concrete should be calculated by the ordinary laws of mechanics, and that, if proper provision was made in the design, it was not necessary to make any surplus allowance for reverse bending moments.

Mr. R. W. Vawdrey, B.A., Assoc.M.Inst.C.E. — He heartily agreed with the author that any such set of rules as those proposed should be looked upon as a means of dealing quickly and conveniently with difficulties that might arise. A set of rules such as that proposed by the London County Council should be used for checking for safety any design which was put before an authority, but, if in any particular case a competent designer wished not to evade the rules but to go more accurately into details, taking into account the circumstances which perhaps were not covered by the rules, it appeared to him essential, if they did not wish to prevent new and progressive design, to allow any designer to substantiate any position which he wished to make, or otherwise design in this material, which was in a very fluid condition. The question of design was continually changing; our methods have changed very considerably in the last two or three years—if no allowance were made for further progress the conditions which obtain at this moment were likely to be stereotyped. If these formulæ are to be adhered to accurately in all cases it appeared to him, as he said before, the whole business was stereotyped in its present condition at this moment.

With regard to some of the details of the paper, it appeared to him that the paper might be summed up into an argument against the use of continuity. Well, he certainly was under the impression that one of the chief advantages of reinforced concrete was the ease with which continuity could be obtained. The author admitted that steelwork could be made fully continuous. He utterly failed to see where the greater difficulty was in making reinforced concrete continuous. The author appeared to treat the bending moments over a support as one which could only be resisted with difficulty. He himself could not see the slightest difference between the resistance of a bending moment over a support and that in the middle of a beam. It appeared just as reasonable to suggest that it was impossible to check in the middle of a beam a bending moment which exists, as it is to say there was any difficulty or danger in resisting a bending moment over a column. If the author suggests that the bending moment over a column might be insufficiently guarded against, and therefore more would be thrown on the centre of the beam, of course that is so; but it was no more likely that the bending moment would be insufficiently resisted over a support than that the bending moment would be insufficiently resisted in the middle of the beam, and he thought, therefore, that it would be just as reasonable to design a floor in which the whole of the bending moment was collected at the point of support, and therefore taken away from the centre of the beam, as to design a building in which the whole of the bending moment was thrown on to the centre of the beam and little or no opportunity for resisting the bending moment at the support was made.

The author did not appear to lay sufficient stress on resisting the greatest moment which must occur at any point. If the floor were designed as he suggested with, say, a bending moment of $\frac{WL}{40}$ at the support and the greatest moment in the middle of the span, if that floor were loaded throughout, however great the bending moment which had been allowed for in the centre of the span might be, the $\frac{WL}{40}$ at the support would not be sufficient to resist the bending moment which actually occurred at that point, and therefore cracks would take place at the support, and the whole load would be thrown on the middle of the span. In that case it would be necessary to take the bending moment at the middle of the span as $\frac{WL}{8}$, and, therefore, in every case it appeared to him that the full bending moment which will occur at that point must be taken.

He would suggest that all the Rules of the London County Council are very fair for use, as, more or less, general rules of design, and that any difficulty such as those raised by the author in his paper would be fully met if a clause were inserted in the Regulations that any designer who wished to go beyond the Rules in any particular must satisfy the authorities that he has good reason for acting as he proposes to do.

Mr. Ewart S. Andrews, B.Sc. (Eng.). – He would like to point out that the argument and theory upon which this $\frac{WL}{12}$ was obtained were based upon the assumption that a beam was a constant section throughout its length, or, rather, it was based upon the assumption that it had a constant moment of inertia, and that assumption, of course, did not hold in the case of a beam in which you have the reinforcement at the bottom and then bent up and used over the support. That, in the actual case, more nearly approximates to the case of a fixed beam of uniform strength. In that case you would have the point of contra-flexure of the beam at one-fourth of the span, instead of at the point, 2·11, of the ordinary assumption. And that has the effect of increasing the bending moment at the end and bringing it up to $\frac{3WL}{32}$.

In the consideration of any formulæ of this kind, it seemed to him that we want to try and get some more evidence upon the subject of what exactly are the bending moments at the fixed end of the fixed beam; because, if we are going to attempt to make our calculations to allow for a fixed end and to avoid the development of the cracks which must occur if no reinforcement is made, then, if we calculate satisfactorily, say, for the $\frac{WL}{12}$, if that is the bending moment, there is no reason to worry about the $\frac{WL}{8}$ which would occur if we had cracks, because there is no reason to suppose that the cracks will ever develop.

It seemed to him that the treatment on page 12 of the Paper was one of very great importance in this matter, because, of course, if you are unable to make any allowance at the end, if the slab is, as it were, at the wrong side to be of any use to you in the resisting of compression, then you really have a beam in which you have got double reinforcement, and, as the author says, on the ordinary assumption where you allow for the concrete in compression, you have to have very much more steel in the compression side than in the tension side. It seemed to be only reasonable that in such circumstances we ought to be able to throw over the compressive resistance of the concrete altogether and merely consider our beam as made up of two steel sections which are held at a suitable distance apart.

Mr. Morgan E. Yeatman, M.A., M.Inst.C.E., M.Am.Soc.C.E. — He heartily agreed with the author's general conclusion as to not being bound by formulæ without any discretion, and, if it is likely that formulæ will be officially adopted, it is a very good thing that we should have a discussion on them and see what is likely to be put before us.

Regarding the columns, he entirely agreed with M. Behar, and thought general practice was unanimous on that—that where only a moderate compression, not exceeding 600 lb., or perhaps some lighter load, as 500 lb., was allowed on the concrete, the whole section of the concrete was taken into consideration. When, on the other hand, a higher compression was allowed in consideration of the increased strength due to hooping, only the section inside the hooping could be considered; for it was obviously the case that where such columns were tested to destruction the outer concrete scaled right off by the time the inner concrete gets up to its maximum load.

He had occasion, at a recent discussion at the Institution of Civil Engineers, to protest against making too great an allowance for continuity in beams. The continuity is there, undoubtedly, but it must be remembered that the loads were not always on all the spans at the same time. It was rather a difficult matter to calculate it for a large number of spans, but he had worked out for three spans the abutment loads and the maximum moments under different conditions. In the middle span the maximum was $\frac{3}{40}WL$, supposing it to be of uniform section all through and uniformly loaded; that is, at the centre of the centre one when it is loaded and the other two are unloaded.

Now the maximum moment on the end span is WL multiplied by 81 and divided by 800. It is a little greater than one-tenth, and that is obtained when the two end spans are loaded and the middle span unloaded. The maximum moment over a support is $\frac{7}{60}$ of WL, which is obtained when one end span and the central span are loaded and the other end span is unloaded. When all the spans are loaded (which, of course, is the case for that part of the load which is dead-load), the moments over the supports are WL divided by 10. In the middle of the middle span it is WL divided by 40, and at the maximum point on the end span it is WL multiplied by 2 and divided by 25. With the central span only loaded, the maximum moment at the centre is $\frac{3}{40}$ WL. Now that shows that, except in the middle span, you are liable to have moments both at the centre of the span and over supports slightly exceeding WL by 10; but, of course, in all practical cases there is a uniform dead-load, as well as the variable load which may or may not come on the span. On the ratio between those two will depend what the actual maximum is in any case.

When it comes to T-beams, we are certainly landed in a difficulty by the fact that over the supports we have no T to take the compression, and we can hardly provide one in concrete. He thought the alternative of deepening the beam at the support by a bracket was a very good one. Certainly any method which led to the putting in of three times the quantity of steel in compression at the support that you have in tension at the middle of the span was an irrational one, for to use heavy steel reinforcement in compression at about 6,000 tons per square inch was not an economical way of treating the material. If that had to be done, it certainly would be more economical to sever your continuity and provide an ordinary beam with $\frac{WL}{8}$ at the centre. But every practical constructor would agree that, having done that, you would not have increased your strength by severing the continuity. It would be better to leave the continuity for what it is worth, and in practice you would not go far out if you provided for $\frac{WL}{10}$ at the centre and carried about the same amount of metal over the supports.

He agreed with Professor Adams as to the danger of calculating the slabs for T's to the main beams—that is, in the kind of constructions where the secondary beams or joists were much closer together centre to centre than the main beams or girders. In that case the floor slabs span from joist to joist, the tension in that direction was taken by rods and the upper part of the floor was in compression, to whatever amount of compression is allowed, parallel to the main beams and at right angles to the joists. If you make the main beam a T, you put a compressive strain on that slab, which must necessarily be added to the compressive strain which is on the top part of it already. In the bottom part of the slab there is a compression put on by the use of the T which negatives the tension that there would be in the bottom part of the slab, and at the top over the abutments the strains are in opposite directions; but at the centre of each small span necessarily the maximum compressive strain, both from action as a T-beam and from action as a slab, is at the top extreme fibres. We find, in point of fact, that those two compressions will be added together; therefore, if you are to count it as both, you must allow for the maximum strain to be safe under the two compressions.

The whole thing pointed to the conclusion that the author had arrived at—namely, that it was both difficult and dangerous to lay these things down by hard and fast formulæ. They ought to be designed by people who understand them.

Mr. Herbert E. Steinberg, Assoc.M.Inst.C.E. — It seemed to him, first of all, desirable to clear up the difference between a beam with fixed ends and a continuous beam. The author rather fell into the error of treating them both as though they were one and the same thing. The question of continuous beams was one that could be solved with almost the same accuracy as the ordinary beam with free supports, having a bending moment of $\frac{WL}{8}$. The only point on which there could be any doubt was as to whether it is permissible to assume the beam as having a constant moment of inertia. There were, however, experiments—chiefly German experiments—which proved that this was quite a reasonable working hypothesis; and, therefore, once you adopt a constant moment of inertia, you could calculate, as one of the speakers had already calculated, the bending moment for a continuous beam of any number of spans under any condition of loading.

The Regulations proposed by the London County Council lay down fairly clearly what to do with beams when the loading is evenly distributed; but there was also the difficulty of a continuous beam in which the loads were applied at points along the beam and not distributed uniformly throughout the length. The various spans of the beam may also be unequal. Occasionally one came across a building where the columns stop at the first floor and the first floor beam has to carry point loads from two columns. It was still a continuous beam, but it would be ridiculous to apply bending moments of $\frac{WL}{12}$, or WL over any factor done by guesswork. The only thing to do was to calculate it properly.

With regard to beams with fixed ends, the amount of fixity usually depended not upon the beam but upon the part of the structure which takes the end of the beam. If there is a sufficient height of brickwork above the built-in end of the beam a negative bending moment, or a counterwise bending moment, may be developed sufficient to neutralise the clock-wise bending moment in the beam, and, therefore, it might realise a truly fixed end $\frac{WL}{12}$, in which case the bending moment at the middle of the span would then be relieved until it became

$\frac{WL}{24}$. Even in this case it would not be prudent to adopt $\frac{WL}{24}$; and, therefore, if you really want to make the best job of it, you would calculate the bending moment at the ends of the beam as though they were perfectly fixed, and you would calculate the bending moment in the middle of the beam as though the ends were imperfectly fixed.

The London County Council proposal to lay down a bending moment of $\frac{WL}{12}$ was, as a general rule, a very reasonable proposal. It was not nearly so severe as the German regulations, where, if you could not calculate the bending moment accurately, you are forced to take the $\frac{WL}{8}$ over the support and $\frac{WL}{8}$ in the span. The London County Council bending moments allowed a good deal more latitude than that.

There is a fundamental fallacy into which, it appeared to him, at any rate, that the author and one or two speakers had fallen, viz. :—That by taking a bending moment of a certain amount at one point in a beam you thereby reduced the bending moment at another point in the beam. You did nothing of the sort. The bending moment is not a thing over which you have any control at all; it is a thing which depended on the loading of the beam and the span of the beam. What you have to do is to try and estimate it by calculation as accurately as possible. Because you may have determined, with superior wisdom, that the bending moments in the spans shall be $\frac{WL}{12}$ and over the support $\frac{WL}{40}$, the bending moment over the support will not change, nor will the actual bending moment in the span alter. If you put in sufficient steel for $\frac{WL}{40}$, then, as the bending moment you have taken is not enough, the steel will be over-stressed and the concrete will crack, and your bending moment at the middle of the span will then be greater than you assumed.

Then, again, it did not appear to him to need any laborious proof that it was more safe to take $\frac{WL}{12}$ than $\frac{WL}{40}$; $\frac{WL}{8}$, $\frac{WL}{6}$ would be still more safe.

One speaker gave the loading of the bending moment for various conditions of loading on a girder of three spans with the ends free, and he said the bending moment for the dead-load which cannot vary, from span to span, as a maximum of $\frac{WL}{12{\cdot}5}$ in the end span. $\frac{WL}{10}$ over the first support, and $\frac{WL}{40}$ in the centre span. Now a super-load may exist on the centre span only, the end span only, or it may exist on the two end spans; you, therefore, will have to calculate those three cases, take the worst possible bending moment from each case, add that bending moment to the bending moment of the dead-load, and then you will have done your work as accurately as you possibly can. And if you calculate your reinforcement according to those bending moments, you have made the best possible job you can of the work.

The author said, if you do anything like that, you are led into great difficulties with your bending moment over support and are almost forced to put spirals in the gussets, but I do not think you are. How is it that in Germany, where the official bending moment over supports is $\frac{WL}{8}$, and Considère's spiral patent still holds, an enormous amount of reinforced concrete is done to the official regulations? If it were impossible to design with $\frac{WL}{12}$ over the support, it would be perfectly prohibitive with such a regulation as $\frac{WL}{8}$.

With regard to the method of putting in spirals in the lower side of beams near the points of support, the author apparently considers this to be a good and satisfactory method, and, as an admirer of M. Considère, the inventor and patentee of the spiral, the speaker fully agreed with him; but it is not essential to do it that way. It might make the best job of it, but the difficulty could be overcome in other ways.

With respect to the compression of the concrete in the slab, apart from the fact that the compression in the slab due to the bending moment in the slab existed only for about one-third of the way down, there was also another factor, and that was that, quite apart from how you considered it, immediately a slab passed over the beam it was prevented from deflecting in the direction of its span, and, therefore, there was no bending moment and no compression in the slab at that point, and that action extended for a considerable area on either side of the beam; so that even although on paper the arrangement of the beams might look as though the slab was doubly compressed, in practice it could not possibly be so.

There were one or two points on which he agreed with the author, and first with regard to posts. It was quite reasonable that the cover of the concrete should be recognised as contributing to the strength of the post. If you enhanced the value of what you might call the basis stress on the concrete by putting either a series of ties closely together or by a spiral, you could put a larger stress on the concrete. But nevertheless he thought that the large area outside the round or the square core might be taken at 600 lb. per square inch. It inevitably contributed to the strength of the column.

E. Flander Etchells, F.Phys.Soc., M.Math.A., A.M.I.Mech.E., moved the vote of thanks to the author, and said the adoption of reinforced concrete work depended upon two factors, first its cost, and second its safety, and we should seek a reasonable compromise between these two divergent ideals—*i.e.,* we wanted the greatest stability with the least cost.

The British specialists had recommended that in all important cases the bending moments under all possible conditions of loading should be accurately determined, and the beams designed at each point in their length to resist the maximum bending moment which may occur at such point, but for small ordinary floors a sufficiently accurate result would be obtained by using a bending moment of $\frac{WL}{12}$, both over the support and in the centre of the span, for continuous beams.

The Second Report of the Joint Committee of the Royal Institute of British Architects recommended that the bending moments should be calculated on ordinary statical principles, and the beams or slabs designed and reinforced to resist these moments. Where the maximum bending moments in beams or floor slabs continuous over three or more equal spans, and under uniformly distributed loads, were not determined by exact calculation, the bending moments should not be taken less than $\frac{WL}{12}$ at the centre of the span and $\frac{WL}{12}$ at the intermediate supports.

Regarding steelwork, where there is admittedly a greater degree of continuity, in general it may be said that the bending moments at the supports next the end are always greatest, and are there about $\frac{WL}{10}$ and near the centre supports they are nearly uniform at about $\frac{WL}{12}$.

Since the draft regulations were written, we have from America, the land of up-to-dateness, two further reports, or codes, and they both fully support the draft regulations in respect of the bending moments. The Joint Committee representing the American Society of Civil Engineers, and the principal scientific institutions over there recommend "That for beams the bending moment at centre and at support for interior spans should be taken at $\frac{WL}{12}$ for both dead and live loads." Then again, it goes on to say, "For spans of unusual length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads. Even if the centre of the span is designed for a greater bending moment than is called for by the previous paragraph, the negative moment at the support should not be taken at less than the values there given." Furthermore, it is stated that "In the design of T-beams acting as continuous beams due consideration should be given to the compressive stresses at the support."

The New York Code, which came into force this year, states that "The bending moments at centre and support for beams or girders continuous over two or more supports shall be taken at $\frac{WL}{12}$." The Regulations of the Royal Prussian Ministry say that "If conditions at support produce restraint and continuity of slabs and beams, the bending moments which appear at those points must have reinforcement placed near the upper stressed surface in proportion to the bearing area."

"If a continuous beam or slab cannot be computed, or, in regard to the latter, if no restraint is certain at a beam or wall, then, with equal panels and uniformly distributed load, the moment is not to be taken less than $\frac{WL}{8}$ over the supports or than $\frac{WL}{10}$ at the centre of panels."

It has been said that the draft regulations under discussion would prohibit the use of reinforced concrete; that it would kill the trade. In reply, he asked whether the reinforced concrete trade was dead in Germany? Was it dead throughout the United States?

With regard to the breadth of T-beams, we are also told that the regulations will militate against the use of reinforced concrete, but that does not seem probable when we bear in mind that the particular regulation referred to was suggested by the Committee of Specialists themselves. They unanimously and on their own initiative recommended that the width of the area under compression in T-beams should not be greater than fifteen times the thickness of the slab or flange.

The New York Code, revised this year, states that the breadth of the compression flange of the T-beam should not exceed twelve times the thickness of the slab plus the breadth of the rib. Therefore it would appear that the regulation under discussion of fifteen times the thickness, as recommended by the specialists themselves, is apparently good practice, and also more or less accords with international practice.

Take the ratio of the breadth of the flange to the span of the beam, the regulations of the Royal Prussian Ministry say you shall only take one-sixth of the span as the breadth of the rib. The New York Revised Code gives the same figure. The American Joint Committee are a little more lenient, they say you may take one-fourth of the span.

The L.C.C. Regulations allow twice the width of the Prussian, twice the width of the New York, and permit a breadth of one-third of the span. The complaints seemed to arise irrespective of the weight of evidence.

Mr. Béhar. — He was of opinion that regulations framed for the purpose of becoming official and legal should not impose formulæ, but only principles, enabling formulæ to be established, or, in any case, if formulæ are given, he was of opinion that these should be given as examples of what would be required, but that these particular formulæ alone should not actually be imposed.

Mr. Etchells. — If principles only were given, very few architects, surveyors, magistrates, or builders would be able to truly affirm whether the statical requirements of the regulations had been complied with. Basic principles might please the professor of engineering, but would they help the busy builder who had to put up buildings in a hurry?

Mr. Béhar. — He was of opinion that an engineer or architect who might be called upon to apply Regulations concerning reinforced concrete should have a sufficient knowledge of the laws of mechanics to be able to deal with the various problems which he may have to study, and to oblige him to follow certain formulæ would practically imply that he was incapable of exercising proper control of any scheme or problem in reinforced concrete which he might have to consider.

Mr. Etchells held that such a conclusion was unjustified. The architects and engineers were consulted before any regulations were sent forward to the allowing authority. Then, since the architects and engineers were consulted before the regulations were made, such consultation effectively admits their competency, not only to carry out the works, but to criticise the draft and to participate in the work of law-making.

He maintained also that it would be easier to see whether a particular design complied with certain stresses, than it would be for the Building Authority, or any officers of the Building Authority, to discriminate as to who were competent and who were not. That would be a very arduous task, and a very difficult task, and one open to very grave abuses, and it is much easier to judge a plan than to judge a man.

The Lecturer Replies.

Mr. Béhar.—Professor Adams had stated that the moment at the point of support in the case of continuous beams was not always greater than $-\frac{WL}{12}$. This assertion was correct, but what he (the lecturer) had endeavoured to point out was that in reinforced concrete, by reason of the assumptions which had to be made, to apply the formulæ of continuity, we must apply at all the points of support the greatest bending moment which we have found for one of the supports, and this moment is always greater than $-\frac{WL}{12}$.

In order to throw more light on this subject, let us consider a reinforced concrete floor in which we have to study a series of twelve continuous beams. He supposed Professor Adams would agree that it would not be wise to apply the continuity to the twelve spans, but would require the latter to be divided into a certain number of sections, for instance, three sections of four spans each, the continuity being applied only to each of these particular sections. We have therefore a section of three beams resting on five points of support.

In order that we may be able to apply the theorem of Clapeyron, we must assume that the points of support 1 and 5 are equal to 0, and if we are dealing with equal spans and equal loads we shall have, for the other supports, moments equal to :

$$B_2 = B_4 = -\frac{3}{28}WL \quad > \quad -\frac{WL}{12}$$

$$B_3 = -\frac{2}{28}WL \quad < \quad -\frac{WL}{12}$$

On thirteen points of support we have, by assumption, the following :

Supports 1, 5, 9	Moments $= 0$
Supports 2, 4, 6, 8, 10, 12	Moments of $-\frac{3}{28}WL$
Supports 3, 7, 11	Moments of $-\frac{2}{28}WL$

Owing to the peculiar nature of reinforced concrete constructions in which the supports of the beams are constituted by other beams or posts in the same material, it was obvious that the free supports did not exist, and he presumed Professor Adams would not consider the points of support 1, 5 and 9 without reinforcement to resist negative bending moments, the value of which, however, is in reality unknown. Moreover we may take as the starting point of a section of four spans any particular point of support, so that we would find ourselves obliged, by measure of precaution, to assume that the greatest moment found for one of the supports should be applied to all the supports.

If the total length of the twelve spans had been divided up into groups of three beams the greatest moments on the supports would have been $-\frac{WL}{10}$. Therefore he was right in saying that in continuous beams it was necessary to provide, at the points of support, a moment greater than $-\frac{WL}{12}$.

Professor Adams is of opinion that in the principal beams the compression of the slab must not be taken into consideration, for the reasons which he had given. Mr. Vawdrey had answered upon this point. He would take the opportunity, however, of the question raised by Professor Adams, to explain more clearly his method of reasoning concerning this matter. Concerning pages Nos. 6 and 7 of his paper, which had appeared rather obscure to some of the members of the audience, amongst others to Mr. Etchells, who is fathering the Rules which are at present under discussion, let us consider a slab supported by secondary beams in reinforced concrete (*Fig.* 1), and let us examine a square portion of this slab having 1 sq. in. in section, of which the centre is situated at the point of intersection of the two axes passing at the middle of the span of the slab and at the middle of the span of the beam. The upper face of the element A under consideration will be submitted to an effort of compression F, owing to the flexion of the slab. This effort of compression will be normal to the direction of the secondary beams, and owing to the flexion of the secondary beams the element A will also be subjected to an effort of compression directed in a parallel direction to the secondary beams. Therefore the resultant force R of these two forces F and F′ might be greater than 600 lb. per sq. in., inasmuch as the force F is already itself almost equal to 600 lb., if we assume that the slab is free to bend on the entire span between the secondary beams without being influenced in any way by the flexion of a portion of this slab working with the secondary beams, which, in fact, would be exaggerated.

FIG 1

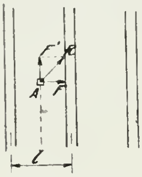
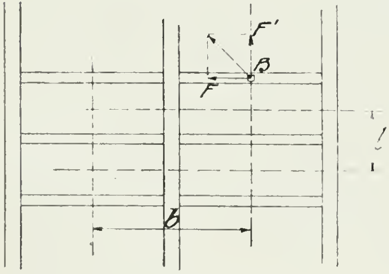


FIG 2



Let us now examine the compression of the slab when the principal beam comes into play (*Fig.* 2). Let us consider an element B, having 1 sq. in. in section and situated on the upper face of the slab, at the middle of the span of the secondary beam, which is nearest to the middle of the principal beam.

This element B receives a compression F coming from the flexion of the secondary beam, and directed normally to the main beam. This element is not influenced in any way by the slab, because in reality the entire area B is deflected with the beam. From the upper beam the element receives a compression F′, acting in a parallel direction to the principal beam. The resultant R of the two forces F and F′ will be generally less than 600 lb. per sq. in., because the forces F and F′ will be usually much less than 600 lb. per sq. in. if the whole of the width 1 of the secondary beam and *b* of the main beam are taken in consideration.

He concluded by what precedes that for the principal beams the regulations should be less stringent in the determination of the width *b* to be taken into account in the calculations. Mr. Vawdrey held the same opinion as Professor Adams in saying that the formulæ of the regulations should not be imposed, but should simply be set forth as a guide. He was entirely of their opinion on this point.

Mr. Vawdrey also stated that he did not see why continuity should not be applied to reinforced concrete. He (Mr. Béhar) did not say that the method of continuity should not be applied. He had simply drawn their attention to the fact that in his opinion the method of continuity was further from the reality in reinforced concrete than in steel construction, unless we assume at all the points of support the greatest bending moment found for one of these points. Continuity finds a better application in bridge work, especially when the points of support of the beams are constituted by a series of piers, the ends of the bridge being supported by masonry abutments.

We must not lose sight of the fact that the theorum of Clapeyron is based on three fundamental assumptions :—

(a) That the moment of inertia of any section of the beams is constant.
(b) That all the points of support are on the same level.
(c) That the points of support are simply represented by lines, or, in other words, a sharp edge at each point of support.

The first of these conditions is never ensured in reinforced concrete beams.

Concerning the third of these conditions, although this might be assumed for secondary beams resting upon principal beams, he failed to see how this could be attained when the beams are supported by pillars, the scantlings of which in certain cases reach 24 in., especially when these posts are supporting other posts above transmitting heavy loads. In such cases we would have a perfect fixation, and $-\frac{WL}{12}$ at the support could be imposed with $\frac{WL}{24}$ in the middle of the beam—that is, if we wish to be in perfect accordance with the laws of mechanics. He was not of opinion, however, that this should be done, as this would lead them to have an insufficient amount of steel in the middle of the beam, which would cause the latter to be weak, if by any chance the amount of fixture at the points of support had been over-estimated.

Concerning the assumption (b) he would simply say that it might happen that in floors supported by a certain number of pillars some of the latter may sink more than others, owing to defective foundations, and if this were to happen it is possible that the span of one of the beams instead of being 1 might become 2 × 1, owing to the sinking of one of the intermediate supports. Unequal sinking of any pillar is, however, less likely to happen in reinforced concrete than in any other material, owing to the monolithic nature of the construction.

He would deal with Mr. Vawdrey's argument against the points of support with a moment of $-\frac{WL}{40}$ at the same time as he answered Mr. Etchells on this point. Concerning the gussets in the beams, Mr. Vawdrey was unwilling to assume that these should be considered as cantilevers, but simply as an increased height of the beam at the points of support. He (the lecturer) held the opinion, on the contrary, that if the gussets were properly reinforced we may treat these as cantilevers without endangering the construction, and this had the advantage of bringing about an economy in the construction.

In reply to Mr. Steinberg, who stated that the moments of $-\frac{WL}{40}$ and $\frac{WL}{10}$ or $-\frac{WL}{24}$ and $\frac{WL}{12}$ are only guesses—namely, that we assumed one or the other at the supports or at the middle of the beam, and that the other corresponding moment was determined by calculation. These, however, were not guesses, but assumptions, which were made in the same manner as those which would have to be made in calculating continuous beams when these were assumed, in order to apply the theorem of Clapeyron, the various hypotheses which he had mentioned in his answers to Professor Adams and to Mr. Vawdrey.

Regarding compression at the points of support Mr. Steinberg said that all we had to do was to increase the width and height of a beam or secondary beam at the point of support, in order to take up the compressive resistance at these points. He probably forgets that floors in reinforced concrete depend upon the requirements of architects more than upon the engineer designing the scheme. Very often the height and width of the beams have been fixed to satisfy other conditions which have nothing to do with the calculations. What would he do in this case to resist high compressions, with a stress of steel limited to 6,000 lb., or even to 4,000 lb. only per sq. in.

It is necessary that, although we may adopt means to increase the security of the construction, the economical side of the question should not altogether be overlooked, and it would appear to him that Mr. Etchells, who, he understood, was the compiler of the proposed Regulations, is inclined to neglect this side of the question on the plea that this does not concern him.

He understood, and he took it that this is not far from the truth, that Mr. Etchells in order to elaborate the proposed Regulations of the London County Council had gone through all the former regulations, both English and foreign, and that he had taken in these all the most severe conditions, which, by the way, have also been criticised in other countries, and that he had introduced the latter in the proposed Regulations which we are now discussing. He must, however, state in all fairness to Mr. Etchells that when he has found the requirements of the foreign regulations were really too stringent he has rendered them somewhat less stringent in favour of reinforced concrete. For instance, we are authorised to assume 17,000 lb. in tension for the steel and $n=15$, whereas other countries authorise much less.

In certain cases, however, the advantages offered are only apparent. For instance, Mr. Etchells had told them in the discussion of the paper that the rules of the L.C.C. specify $-\frac{WL}{12}$ at the supports, whereas the Prussian regulations stipulate $-\frac{WL}{8}$.

He would like to know how those engineers who have to apply these regulations overcome the difficulty of the compression of the points of support which he had mentioned above. Mr. Etchells, however, had forgotten to tell us that these same Prussian regulations authorise a working stress of the concrete in tension, and attribute to the latter a stress equal to two-thirds of the crushing stress of the concrete in tension, and as they assume that the resistance to crushing in tension is equal to 228 lb. per sq. in., the application of these rules would lead us to have a width *b* of the slab working in tension, at the same time as the beams at the points of support, at an average stress of 120 to 140 lb. per sq. in. On this account the bars in tension at the points of support would represent a section of steel, certainly much less than the section which we obtain by the application of the formula $-\frac{WL}{12}$ of the proposed Regulations of the London County Council.

Concerning the compression at the points of support, we do not know whether the Prussian regulations stipulate that the section of steel should be calculated by applying to the latter 15 times the working stress of the concrete applied to the centre of gravity of the bars, whereas the proposed Regulations of the L.C.C. actually adopt this view.

To conclude his answer to Mr. Etchells' remarks, the aim of his paper had not been to try to urge that certain methods of calculating a moment for a beam in reinforced concrete should be adopted in preference to any other method, but to show that a greater safety could be obtained by providing at the midde of a beam a bending moment higher than $\frac{WL}{12}$, or at least equal to this value, and to show if the method of calculating beams as continuous may be sometimes dangerous. It is precisely because by this method one is led to provide in certain cases at the middle bending moments which are smaller than $\frac{WL}{12}$, whereas at the points of support, if the designer of a scheme provides reverse bending moments less than $-\frac{WL}{12}$, the danger is not so great on account of the table *b* of the slab, which is working in tension at the same time as the beam. In fact, to a certain extent this view was confirmed by Mr. Etchells himself, since he would have no hesitation in considering a beam in reinforced concrete as being free upon its supports, even if it were resting upon another beam or a pillar, on condition that the bending moment at the middle should be taken equal to $\frac{WL}{8}$, but it is to be noticed that, for greater safety, he advises in this case to provide at the points of support a few small bars in the upper portion of the beam.

It appeared to him that if Mr. Etchells were willing to accept that method he did not differ very much in opinion from those who thought that the safety of a beam was not impaired by adopting $\frac{WL}{10}$ in the middle and $-\frac{WL}{40}$ at the supports, or any other similar assumption, on condition, of course, that as far as the middle of the beam was concerned they always remain above $\frac{WL}{12}$.

He regretted that owing to the length of the debate it had not been possible for him to reply individually to each of the remarks of the other members.

NEW WORKS IN CONCRETE

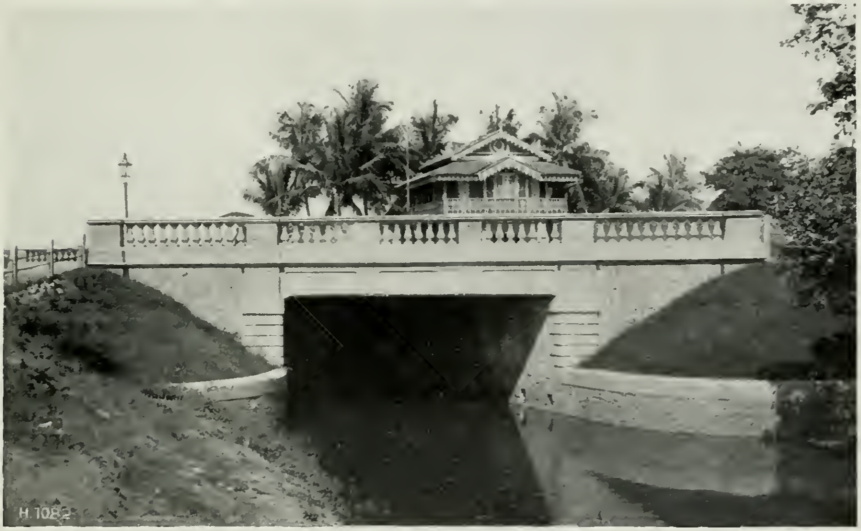
AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE BRIDGE IN COLOMBO, CEYLON.

We have given examples from time to time of different applications of reinforced concrete in India, China, and Japan, and we now have pleasure in showing an illustration below of a bridge which has recently been constructed in that material for the Municipal Council of Colombo.

It is the first bridge of its kind to be erected in Ceylon, and was constructed entirely with coolie labour. The bridge spans the old Dutch canal, along which boats carry rice, tea, rubber, etc., from the low country estates to the Port of Colombo, where the produce is landed to the ships bound to all parts of the world.



View of finished Bridge.

Reinforced Concrete Bridge in Colombo, Celyon.

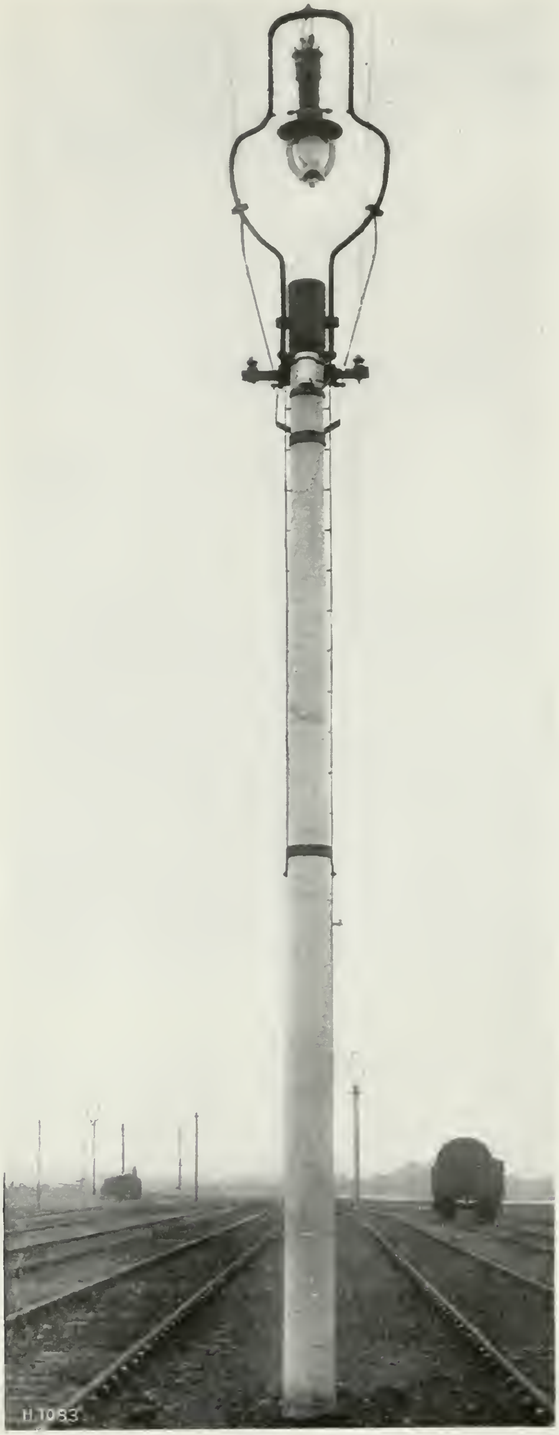
The bridge, which is constructed on the Kahn system, with stone masonry walls, is 100 ft. wide and 25 ft. in span. The cost was Rs. 70,000, or Rs. 28 (37s.) per sq. foot of waterway width. It was designed by Mr. Robert Skelton, A.M.Inst.C.E., Municipal Engineer of Colombo. The above particulars were sent us by Mr. Clive H. Kilmister, Chief Assistant Works Engineer, Colombo Municipality.

REINFORCED CONCRETE POLES FOR THE GREAT CENTRAL RAILWAY CO. AT IMMINGHAM.

The reinforced concrete poles, numbering over 300, which have been supplied to the Great Central Railway Company at Immingham by Messrs. Siegwart, Ltd., of Caxton House, London, S.W., are 30 ft. long. They are buried to a depth of 6 ft., and rest on a concrete "biscuit" 9 in. thick. Attached above is the lamp frame, with an iron ladder for access and attention to the light.

The earth wire for earthing the cradle of the lamp passes through the centre of each column.

The columns are considerably cheaper in initial cost than iron or steel poles, while no painting or other expenditure in maintenance is necessary; moreover, although they



Reinforced Concrete Poles for the Great Central Railway Co. at Immingham.

stand in most exposed situations, no rust or deterioration can take place. Indeed, it is claimed in the light of experience that, as concrete improves with age, the columns will be better and stronger at the end of fifty years than when first fixed.

The external diameter of the poles at the top is $7\frac{1}{2}$ in. and at the bottom $13\frac{1}{2}$ in. They have been designed for a working wire-pull of 300 lb. 2 ft. from the top and at right angles to the poles, in addition to the wind pressure on the pole itself.

For connecting the lamp in series, lead-covered cables laid on the solid system are employed.

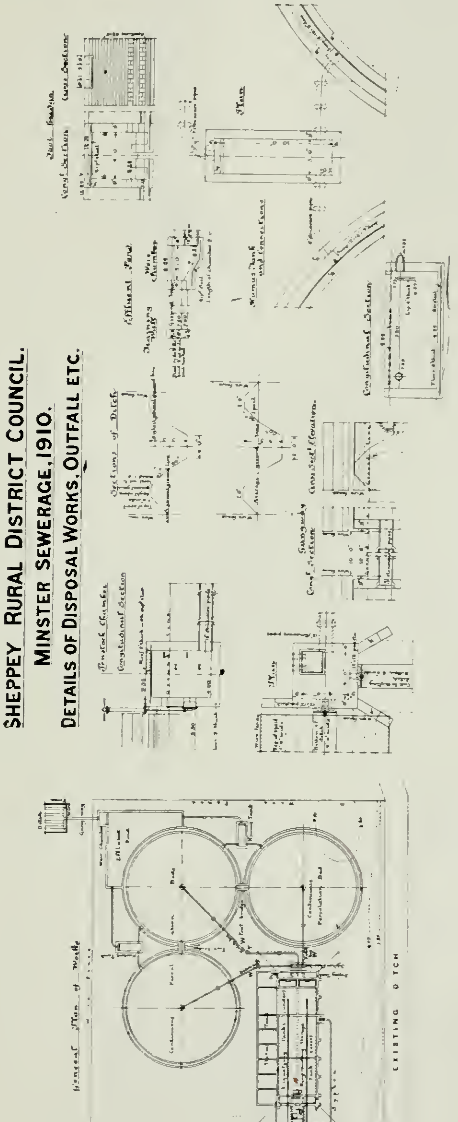
Lamp centres are 25 ft. above ground level, and the cradles are attached by means of sockets grouted on the ends of the columns, as also by side brackets.

REINFORCED CONCRETE LIQUIFYING AND STORM TANKS, MINSTER - IN - SHEPPEY SEWAGE DISPOSAL WORKS.

The design of the sewage disposal works for the district of Minster - in - Sheppey has called for unusual consideration. They have been placed upon extensive flats, lying below high-water level of ordinary tides; and, in order that the final effluent might gravitate from the filter beds to one of the main watercourses, the whole of the works had necessarily to be constructed above the ground surface.

The formation upon which the works have been erected being alluvium, it was considered advisable to reduce the weight so far as circumstances permitted; but the district being liable to floods the question of flotation of the tanks (when empty) had to be considered. In order to avoid vertical dislocation of them during the greatest known depth of water, it was found necessary to increase the mass beyond that actually required in resistance to internal static pressure.

The sewage is brought from the district to the works by an inverted syphon, the outlet bell-mouth of which enters an intake chamber through a cast-iron sleeve forming an annular clear space. By this means the whole structure may rise and fall over the maximum ascertained limit without causing fracture of the syphon at or near the point of connection.


SHEPPEY RURAL DISTRICT COUNCIL.
MINSTER SEWERAGE, 1910.
DETAILS OF DISPOSAL WORKS, OUTFALL ETC.
General Plan of Works
EXISTING DITCH

Details of Disposal Works, etc.

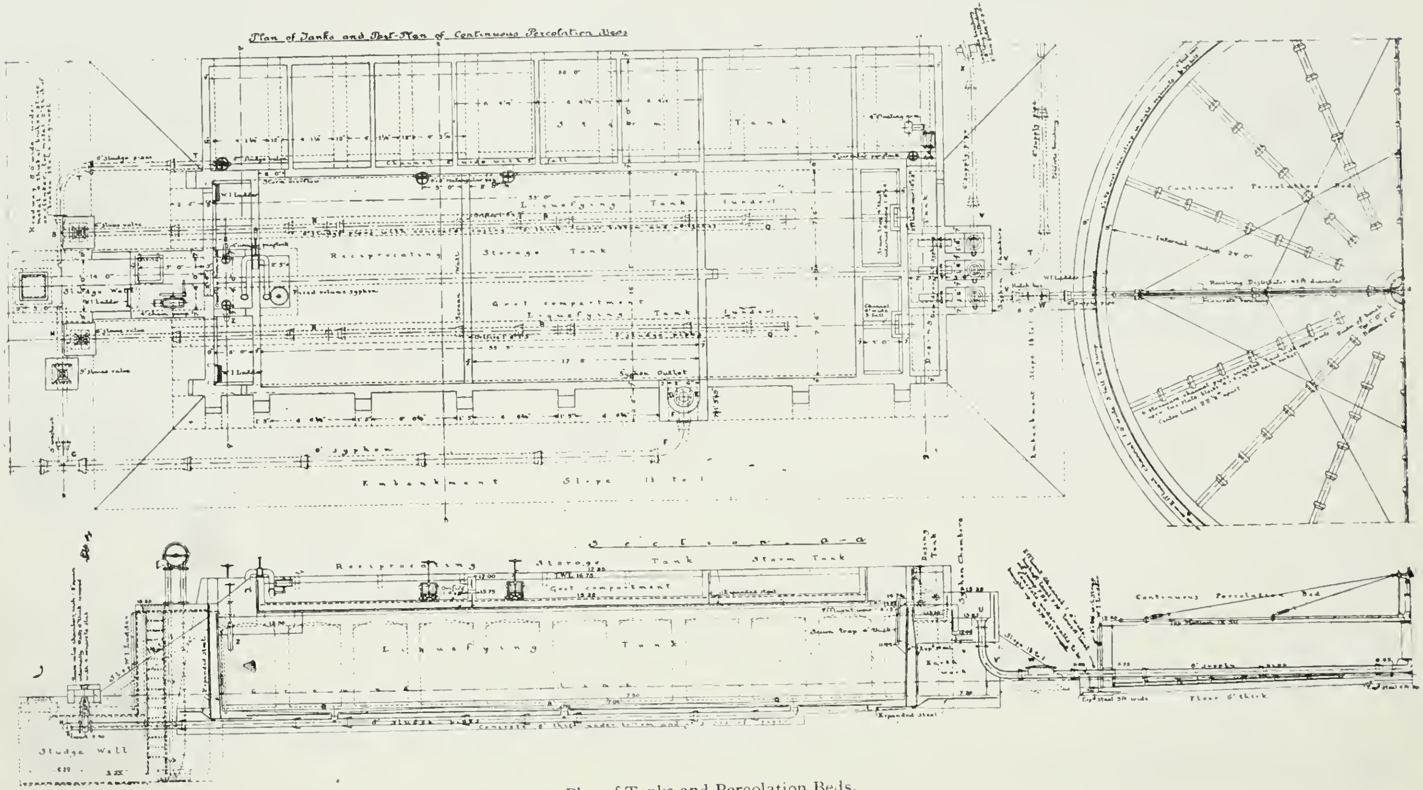
Reinforced Concrete Liquifying and Storm Tanks, Minster-in-Sheppey Sewage Disposal Works.

The tanks are roofed in, the roof forming the floor of a reciprocating storage tank, provided for the purpose of balancing the periodical discharges from two pumping stations controlling the low-level areas of the district drained.

The concrete throughout is composed of three and a half parts of gravel, one and a half parts of sand, and one part of Portland cement, and the walls and roof are reinforced by Expanded Steel, 3 in. mesh.

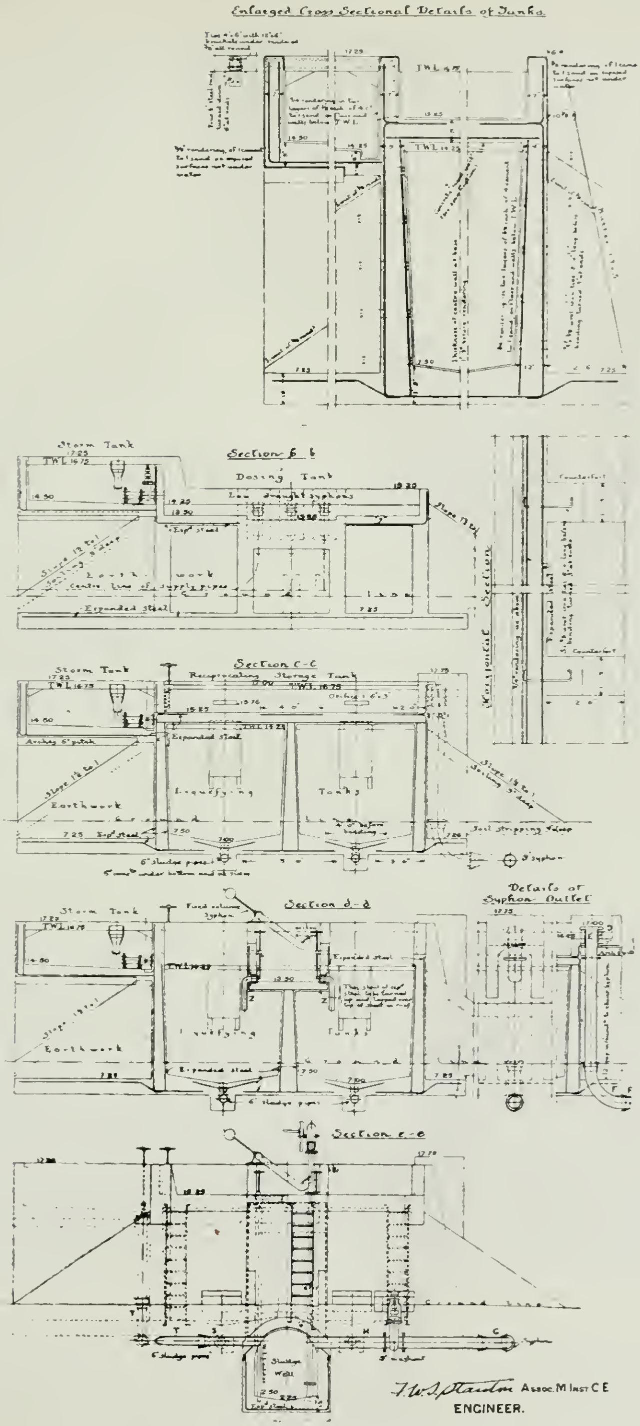
For the purpose of stiffening the structure externally, counterforts were constructed, but in the calculations of flexure of the walls their support was disregarded. The walls being tied in at the foundations and roof they were considered in the calculations as vertical girders under an unequally distributed load throughout the water depth.

Expanded Steel was built in at the water face as well as on the external face; and whilst the latter only is under tensile stress the former was inserted for the resistance of thermal stresses when the tanks are temporarily empty. The steets of Expanded Steel were lapped to the extent of two meshes, the concrete taking up the stress in shear over the laps. The engineer for this work was Mr. F. W. S. Stanton, A.M.Inst.C.E., who designed the tanks.



Plan of Tanks and Percolation Beds.

Reinforced Concrete Liquifying and Storm Tanks, Minster-in-Sheppey Sewage Disposal Works.



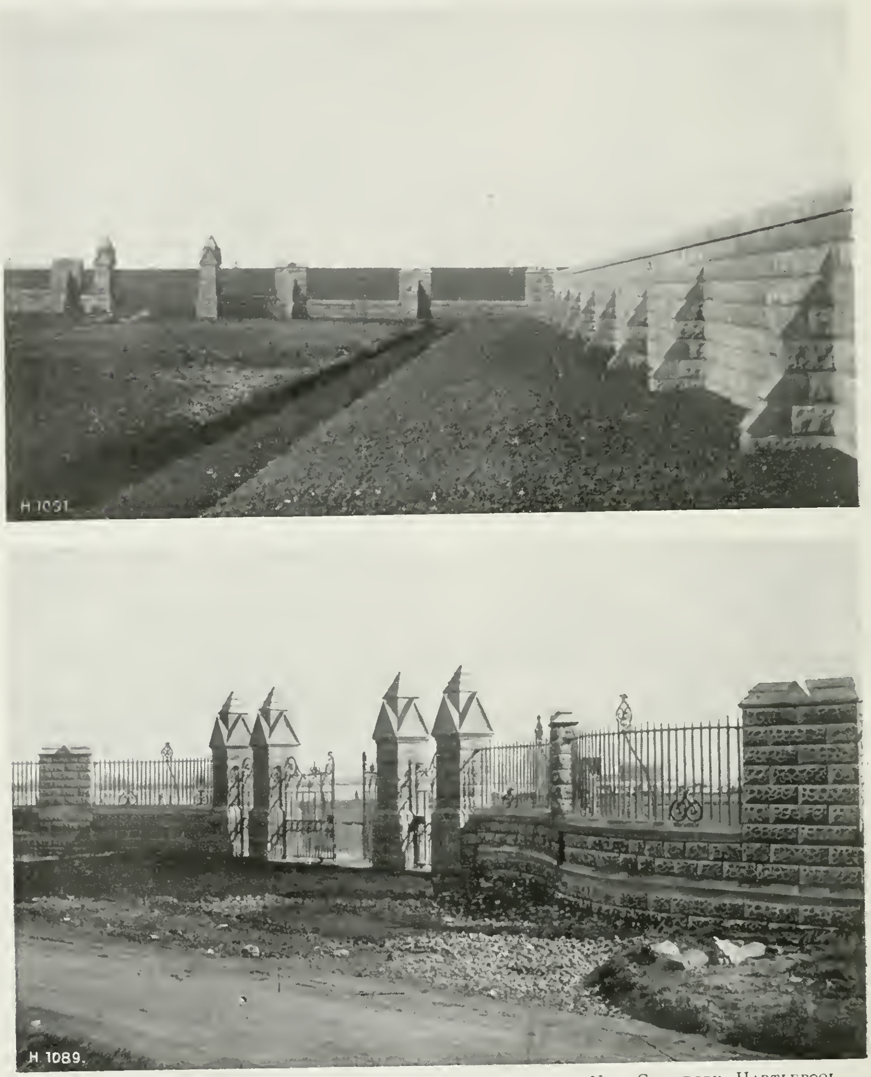
Enlarged Cross-sectional Details of Tanks.

Reinforced Concrete Liquifying and Storm Tanks, Minster-in-Sheppey Sewage Disposal Works.

CONCRETE BLOCK BOUNDARY WALLS AND ENTRANCE GATES TO NEW CEMETERY, HARTLEPOOL.

THE accompanying illustrations show the boundary walls, with main entrance gates, to the new cemetery for the Hartlepool Corporation.

With the exception of the pier caps, the whole of the walls and piers, plinth, coping, panelled blocks, and buttresses were erected with concrete blocks made on the machine supplied by the (U.K.) "Winget" Concrete Machine Company, Ltd., Newcastle-on-Tyne.



CONCRETE BLOCK BOUNDARY WALLS AND ENTRANCE GATES TO THE NEW CEMETERY, HARTLEPOOL.

The total length of the walls is 4,000 ft., and the blocks were made by unskilled labour (unemployed).

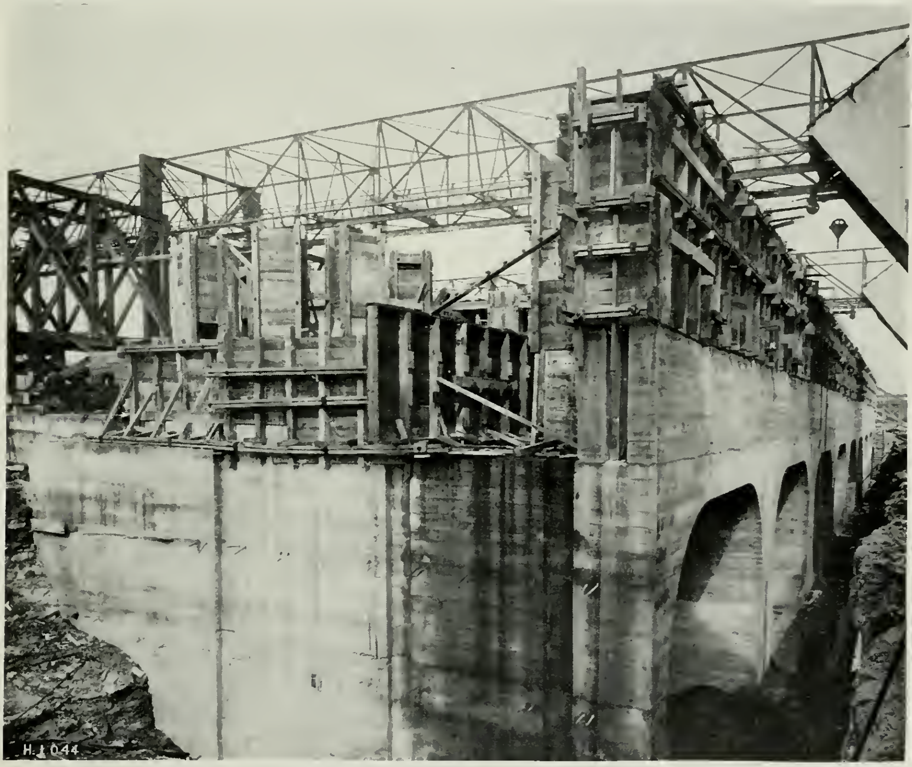
The architect was Mr. H. C. Crummack, A.M.Inst.C.E., Engineer and Surveyor to the Hartlepool Corporation.

CONCRETE IN THE CONSTRUCTION OF THE MISSISSIPPI RIVER'S GREAT HYDRO ELECTRIC PLANT AT KEOKUK.

The great concrete dam across the Mississippi at Keokuk, Iowa, which is being built in connection with the Mississippi River-power development, is now nearing completion. This power plant is designed to develop the greatest amount of hydro-electric power in any single station in the world. Practically the whole course of the river will be turned into electric current, producing 200,000 horse-power.

The project involved the construction of a solid concrete dam across the river, a power-house, and a new lock and dry dock. All these works were carried out simultaneously and with the greatest degree of organisation and extraordinary rapidity.

The dam, including abutments, is 4,560 ft. long, or seven-eighths of a mile. The spillway section is 427 ft. in length, and the height is 32 ft. above the river bed, and its base 42 ft. wide. On top of the spillway are placed 119 steel flood gates, 30 ft. wide and 11 ft. high, supported by concrete piers. These piers are 6 ft. thick, and are built integral with the dam. The piers also support an arched bridge, from which the gates are operated by electric hoists. By manipulating these gates the water above the dam is maintained at a nearly constant level at all seasons.



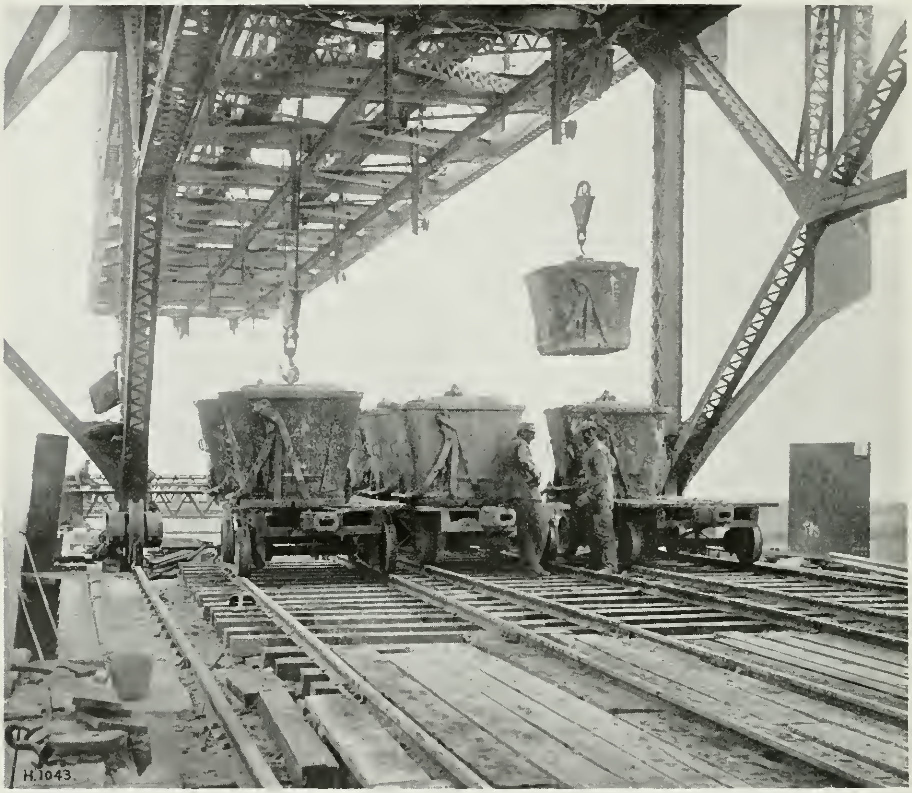
Power House in course of construction.

Concrete in the Construction of the Mississippi Hydro Electric Plant.

The dam is built entirely of mass concrete without reinforcement of any kind, and is locked firmly into the rock bed of the river, it being practically a monolith.

The hydro-electric power-house is 1,400 ft. long and 123 ft. wide, the substructure being built of mass concrete, in which is moulded the water passages and water-wheel chambers. On top of this is built the superstructure—a house of concrete, brick, and steel. The superstructure contains the electric generators, transformers and switchboard, the height of the power-house from foundation to roof being 133 ft. In all, there are thirty power-generating units provided for, each unit consisting of a vertical steel shaft, on the lower part of which are two turbines or water-wheels and on the upper part the revolving parts of the generators.

In order to keep floating ice and logs from entering the power-house an ice fender has been provided with the shore, 2,800 ft. long, built of concrete masonry. To the eye it appears as a solid wall fencing off the power-house from the river; there are large arched openings below the water-level through which the water will find its way to the power-house.



Great crane at work for distributing concrete.

CONCRETE IN THE CONSTRUCTION OF THE MISSISSIPPI HYDRO ELECTRIC PLANT.

It is claimed that no part of the entire construction is founded upon artificial foundations, as solid blue undisturbed limestone rock is everywhere available. There is required in the construction approximately 550,000 cubic yards of masonry, 650,000 barrels of Portland cement, and 7,000 tons of steel.

The concrete-mixing plant for the main dam is a most complete installation. The upper part of this structure is a large wooden bin containing sand and stone. The lower part is a series of concrete arches. Under these arches are the projecting ends of the concrete-mixing machines. The sand and stone are fed down through openings in the bottom of the bins into the mixers, where water and cement are added, and all materials are thoroughly mixed in the revolving drums, which discharge their contents after mixing into large steel buckets resting on cars which run on the track shown in the illustration on this page. The concrete is then hauled out to the point of use on the dam by means of small locomotives.

It is pointed out that the method of building the dam in the river was unique. A coffer-dam consisting of a rectangular timber crib structure, loaded with stone and made watertight by means of clay puddle, was built around the site of a section of the dam, about 400 ft. long, the water then being pumped out of this coffer-dam. In this space, thus made dry, a trench was excavated in the solid rock on which the dam is founded. The piers and arches forming the bridge and the bottom part of the dam between these piers were built, and after this the coffer-dam was removed and another section of about 400 ft. was coffer-dammed and the bridge built in the same way, thus continuing until the bridge is extended all the way from the Illinois shore to the junction with the power-house on the Iowa side. This leaves 119 large openings between the bridge piers, through which the water of the river passes unobstructed. These openings are closed off a few at a time by means of steel gates, and the balance of the concrete part of the dam placed behind these gates, gradually raising the crest of the dam until it has reached its full height.

The concrete-mixer plant is located at the end of the dam and directly in line with it. The substructure consists of four concrete arches, in each of which is one concrete mixer of one and one-half yards capacity. The superstructure consists of the wooden bins—viz., sand and stone bins. Connecting the bins and each mixer are two steel chutes which automatically convey and measure the quantities of sand and stone to the mixers. The cement is dumped into a small hopper which also leads to the mixer, and the water for each "batch" is delivered from a tank located just above each machine. By means of compressed air the charges are unloaded into the mixer, which, in turn, is revolved by an engine driven by air.

There is directly beneath the mixers a double-track railway on which the concrete trains run. Each flat car is loaded with two bottom-dump double-leaf steel concrete buckets. The train stops in front of the mixers, and each mixer, having received its charges from the chute above, discharges its mixture into a bucket directly beneath it. The train with its load moves out on to the dam under the travelling crane, which picks the buckets from off the cars and places them into the forms where they are dumped.

REINFORCED CONCRETE COW-STALL DIVISIONS.

In connection with the advantageous use of concrete and reinforced concrete for farm uses which we are constantly advocating, we show on this page an illustration of reinforced concrete cow-stall divisions at Chipnall for Mr. T. Balfour, Cheswardine Estate, Market Drayton. These were constructed of No. 24 Diamond Mesh Expanded Steel in the centre of 3 in. of concrete. This reinforcement was supplied by the Expanded Metal Company of London and West Hartlepool.

Reinforced Concrete Cow-Stall Divisions.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"The Mechanics of Building Construction." By Henry Adams, M.I.C.E., etc.

London : Longmans, Green & Co., 39 Paternoster Row. Price 6s. net. 240 pp. + xi.

This volume is based on a series of lectures given by the author to a number of science teachers at South Kensington, although a good deal of alteration has been made to the text and many details explained more fully than in the first instance in order to bring the work in line with a student's requirements. A very large scope has been covered, and the book is illustrated by nearly six hundred diagrams, which are clear and explanatory. There is undoubtedly a great need for a good book which deals with the theory of structural design, as there are very few works which deal with the subject in anything like a comprehensive manner, and, in consequence, there is a tendency for students to avoid this highly important subject with the feeling that it is quite beyond their capacities. We can hardly agree with the author that this particular work will meet the needs of the ordinary student, as the fundamental principles are not simply and adequately explained, being, we consider, dismissed with a few remarks which would not enable anyone without a fair knowledge of the subject to grasp their meaning and importance. It must be remembered that if all the fundamental principles are thoroughly understood their application becomes more or less a simple matter, and intelligent design will result; but if the understanding is only partial, then any problem which presents a new aspect will be fatal to the student. The portions devoted to graphics are very well treated and contain many complete and interesting examples which should prove of much value to the designer. We should like to have seen more space devoted to foundation work, as no mention is made of grillages and the methods of calculating the different layer beams, this being of some importance to architects, considering the great use of steel-framed structures. There is a great deal of good material in the volume, and it should prove a valuable addition to the library of scientific works, but we venture to think that it will appeal far more to the teacher than to the student, and thus its use will be limited.

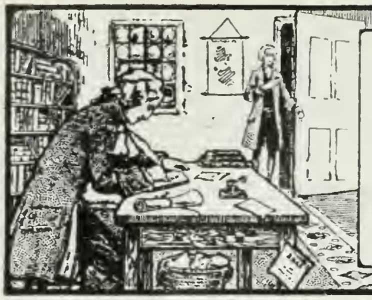
"Concrete Costs," by Frederick W. Taylor, *M.E., Sc.D.*, and Sanford E. Thompson, *S.B., M.Am.Soc.C.E.*

London : Chapman & Hall, Ltd. New York : John Wiley & Sons. Price 21s. net. 8vo. 710 + xii pp.

Contents.—Introduction — Preface — Approximate Costs of Miscellaneous Concrete Work—Approximate Cost Data on Concrete Structures—Approximate Costs of Reinforced Concrete Buildings — Determination of Labour Cost—Task-work in Construction—Proportioning Concrete—Tables of Quantities of Materials for Concrete and Mortar—Cost of Concrete Materials—Excavating and Crushing Stone for Concrete—Handling and Transporting Materials—Labour of Hand-mixing—Machinery Plants for Mixing and Handling Concrete—Labour Costs of Machine-mixing—Forms for Mass Concrete—Arch Centres—Forms for Reinforced Concrete—Tables for Concrete Volumes—Tables of Steel Areas and Quantities—Tables of Times and Costs Bending and Placing Steel—Tables for designing Forms—Tables of Quantities of Lumber for Forms.—Tables of Times and Costs of Labour on Forms—Estimates for Reinforced Concrete Construction.

Everyone will admit that there is plenty of scope for the application of system to building work, so that this book makes a wide appeal in thus devoting attention thereto. Its title would appear to confine it narrowly to one particular branch of building work, but as a matter of fact associated with the carrying out of constructional works in concrete there is much of a general character applying to older ways of building, as, for instance, excavation and handling and transporting material, all of which are duly taken account of in the book, which must also be looked upon as an introduction to the whole subject of scientific management in the building trades.

The authors have our confidence and respect at once for having published that excellent book, first issued some years ago, entitled "Concrete, Plain and Reinforced"; while Mr. Frederick W. Taylor, one of the authors, was a pioneer in introducing, in 1885, scientific methods of management at the Midvale Steel Works, and has been engaged since in systematizing many other companies along similar lines. The *modus operandi* of scientific management appears to consist of making a close analysis of the operations of the workmen, including the time taken in each and every small operation in whole and part, by the process explained by Mr. Frank B. Gilbreth, an American contractor, in the book entitled "Motion Study," which we recently reviewed in these columns, and in which it was explained that by making time studies of the labour of laying a single brick divided into its elementary motions, so as to see whether any of these motions could be left out and the brick laid just as well, he was able to reduce the number of motions in ordinary work from eighteen to five, with very great economy as a result. That is only part of the purpose of this book—it also shows how to use such analysis for ordinary estimating, keeping prime cost and general organization of a job. It might be supposed that because this is written by American authors primarily for American readers, all the prices and measurements being in American units and based on American labour conditions, it would not be of very direct use to English contractors, engineers, and others, but that is not so, for although the high rates paid for labour in the United States have made the problem of increasing efficiency more urgent, yet the data are derived from men of much the same calibre as found in Britishers the world over, and where white labour be employed, though it may not always fit exactly—indeed, the authors do not suggest it will, continually warning us that their figures are more illustrative than to be accepted blindly—it is far and away the best guide to estimating concrete and reinforced concrete work we know of, and we would far rather trust it than any other published data. It is highly instructive and suggestive, and should be closely studied by all who have to carry out building work. The method of estimating by building up prices from unit operations is not new, but, alas! in the building trades it is seldom adopted, and, although builders and builders' estimators in this country may raise all sorts of objections the same as their confrères have done the other side of the Atlantic, the method is nevertheless the right one, and their objections are only the result of ignorance. There are, no doubt, difficulties in introducing piece-work or task-work (the latter consisting of setting a time for a piece of work and paying a bonus if done in the time allowed) generally in the building trades, entailing as it does a laying-out of the exact work to be done by every man at least a day beforehand, yet it should and can be done. We have no space here to summarise all the many valuable hints given by the authors, or even outline the various sections into which scientific organization is divided—the book should be read. We may, however, state that the book contains all the data needed in respect to materials as well as labour, such as the weights of materials, the sizes of sacks, barrels, barrows, descriptions of standard tools, standard centering, etc., etc. There is a good deal of matter about the scientific proportioning of concrete much in extension of the authors' other book, "Concrete, Plain and Reinforced," but here, valuable and correct though the general data are, the tables giving the volumes of concrete obtained from various proportions of ingredients are not applicable to standard English practice, as they are based on cement weighing on the average 100 lb. per cubic foot, whereas 90 lb. is the figure usually and officially adopted in Britain. This means, of course, that English concrete nominally of the same proportions by volume as American concrete really will contain less cement. Let us hope that English cement is proportionately stronger, as we believe it to be, on the average. Many useful tables and diagrams for standard practice, useful to designers as well as contractors and estimators, are interspersed throughout the book, and we feel sure it will react upon current practice to its great improvement, and we hope that in turn practice may react upon the sales of the book so as to warrant a second edition in a year or two which shall be as enlarged and improved as was the second edition of that other standard authority on concrete by the same writers.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Reinforced Concrete for Aviation Buildings.—At the meeting of the Institution of Municipal Engineers, which was held at the London Aerodrome, Hendon, N.W., on June 12, Mr. Horace Cubitt, A.R.I.B.A., read a very interesting paper on "The Structures of the Future in relation to Aviation." Dealing with the necessity which must soon arise—when the industries of aeroplane construction and aerial transit are established as large factors in our national industrial programme—of having permanent aviation buildings such as workshops and aeroplane sheds, Mr. Cubitt said :—

"When consideration is given to the form which this permanent construction will take it is, however, a question whether plain brickwork will hold its own, or whether it will have to give place to one of the more modern forms of construction. As regards the aviation workshops, these, in common with new workshops of other kinds, may very probably be constructed of a steel skeleton filled in with brickwork, or of that new combination of materials, reinforced concrete. Aeroplane sheds, necessarily being, it is presumed, one-storey buildings of moderate height, will hardly lend themselves to steel-frame construction, but it would appear that reinforced concrete is particularly suitable for structures of this character. Experience in connection with the new General Post Office, and elsewhere, has shown that reinforced concrete is particularly suitable for buildings requiring large areas of unencumbered floor space. Therefore it may be assumed that it will in due course be largely adopted for the construction of aeroplane sheds, particularly if it is possible, as I believe now to be the case, to form watertight roofs of this material without asphalte or any other kind of covering. A further important constructional point which does not occur in modern aeroplane sheds, but may be anticipated in future constructions, is the obtaining of large floor spaces with the absolute minimum of columns or stanchions. At present the aeroplanes in use are comparatively small—that is, compared with those we may expect in the future—and it is not, I believe, customary to put a number of aeroplanes in one shed. But in due course, when an enterprise with such a title as the London Aerobus Company is in full swing, each building at the company's depôts will doubtless be constructed to contain a considerable number of aerobuses, and with these vehicles exceeding by many times the size of a modern biplane it will be necessary to keep the floor space almost entirely free from columns. To effect this with due regard to economy will not be easy, but probably by a skilful use of reinforced concrete the problem will be satisfactorily solved."

Turning to the question of the landing-stages which, it seems, will eventually have to be erected in large towns, the lecturer went on to say :—

"If, as seems certain, tall landing-stages are to be erected in districts where land is very valuable, it will obviously be desirable to utilise the space below the landing-stage—*i.e.*, to erect ordinary buildings upon it. But if the site is cleared, and the buildings and the landing-stage are to be erected at the same time, the question will at once arise, Why not form them as one structure? If this is done, it will be obviously a waste of space to provide separate legs for the landing-stage, as such stage can be easily supported on the walls or other enclosures of the building. Thus the aeroplane landing-stages of the future will probably be nothing more than a large and lofty building with a flat roof, on and from which the aeroplanes will alight and depart. As regards the form of construction to be adopted, neither brickwork nor stonework is suitable, if economy is a consideration. Both steel and reinforced concrete possess great advantages over the older forms of construction in the case of the erection of lofty buildings. As to which of these two forms of construction—a properly encased steel frame or a scientifically designed arrangement of steel and concrete—is likely to be adopted, it is hard to say. Probably both forms will find their supporters. A great deal has lately been written in praise of reinforced concrete, but the advantages of a steel frame for a lofty structure must not on this account be overlooked."

Mr. B. Wyand, the Secretary of the Institution, also read a paper entitled "Some Suggestions for By-laws and Regulations in relation to Aviation."

Setting of Cement during Freezing Temperatures.—This has been investigated in a series of tests recently conducted by Messrs. L. Bienfait and H. Bancke. According to *De Ingenieur,* clear water and saline solutions were both used in the mortars tested, and it was found that the salts of sodium and calcium accelerated the process of setting somewhat, even at low temperatures, while the salts of magnesium retarded it. It was observed that setting and hardening proceeded slowly at the low temperatures, provided that the materials were mixed at higher temperatures, and such mortars when placed under normal conditions acquired a strength in time equal to those not subjected to low temperatures. However, after a lapse of thirty days under freezing conditions the mortars showed a compressive strength somewhat lower than that of a seven-day mortar normally treated. In discussing the results obtained the writers recommend the use of clear water rather than saline solution for such conditions.

Jacketing a Steel Water-Tank with Concrete.—The process of jacketing a steel water tank was described in the annual report of Mr. F. J. Fischer, the Chief Mechanical Engineer of the Los Angeles (Cal.) Water Department. For some time the Edendale section has been supplied from a large steel tank having a capacity of 528,000 gallons. While this tank was originally built with a liberal factor of safety, recent unexplained failures of tanks of similar construction led the department to determine to be doubly safe. With this in mind, the capacity of the tank was increased to 593,000 gallons by adding 5 ft. to the height; and the steel shell was then encased in reinforced concrete having a thickness of 5 in. on the inside and 10 in. on the outside—the reinforced concrete walls of themselves being of sufficient strength to withstand the hydraulic pressure with an ample factor of safety. This improvement, Mr. Fischer points out, has meant the replacement of an unsightly black steel tank with a concrete structure simple yet commanding in appearance.

Largest Reinforced Concrete Bridge in the World.—What will be the largest reinforced concrete bridge in the world is to be built (says the *Times Engineering Supplement*) across Tunkhannock Creek, near Nicholson, Pa., by the Delaware, Lackawanna, and Western Railroad. The design is for a double track structure having ten 180 ft. arches with a 100 ft. arch at each end, and totalling 2,230 ft. in length with a *maximum* height above the water of 249 ft. The principal arches are to be reinforced, but they are designed to carry loads independently of the reinforcing. The piers are to be carried down to solid rock, about 95 ft. below the ground surface at the deepest point. The centering to be used in placing the large masses of concrete in position is to consist of hinged trussed arches of steel construction supported by the bridge piers. It was decided to have a ballasted floor construction with a thoroughly waterproof deck, and this would have required a heavy design in steel, which, together with the fact that a material increase in live load would produce a less stress increase in the case of a concrete structure, led to the decision to use this material. It was also felt that the maintenance of a steel structure of such height and length was excessive in view of the liability of the rivets becoming loose and of deterioration being caused by vibration. The cost of renewing such a steel structure, owing to the conditions of height and situation, was estimated to be as great as the first cost.

A Temporary Cement-Wash Protection for Bridge Timbers.—A temporary fire protection for old timbers on a steel bridge where the length of useful service had nearly been reached and the wood badly checked was recently provided on the Boston and Maine Railroad by a wash made up of Portland cement, plaster of Paris, and

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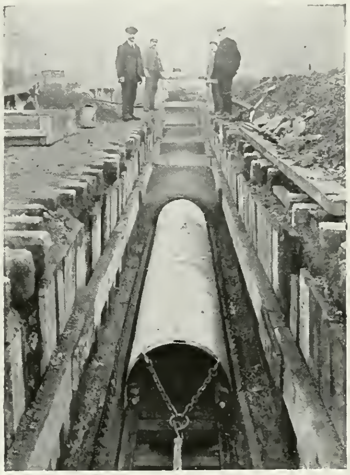

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sand. The bridge was a deck structure, located at such a distance from the station or houses where attention might be attracted by a fire burning on it, that some form of protection was considered advisable during the period of one or two years remaining in the life of the ties. There were wide and deep cracks in some of these ties, and as sufficient experience had not been had at that point with the use of fire-resisting paints to give assurance of their effective service, experiments were made in filling the cracks with various compositions, and also covering the entire top surface of the tiles and guard rails. The first experiment was with cement and water alone, but this plaster cracked and came out of the cavities in the tiles so as to afford very little protection. A mixture of lime and cement and sand was then used on another portion of the bridge, and was found to give better protection. Finally a mixture of Portland cement, plaster of Paris, and very fine sand was tried, this being mixed so thin that it would run in and fill all the cracks in the tiles and guard rails. It was washed over the entire surface of the timber. This form of protection proved effective in stopping fires on the bridge for a number of months, but was considered only as a temporary expedient. The expense of the application was about the same as that for the application of a coating of fire-resisting paint, but the durability was considerably less than that of a paint. Since this work was done the fire-resisting paints have been investigated further, and found to be sufficiently serviceable as a protection. They are believed to be preferable where the protection is desired to last for any considerable period.—*Engineering Record.*

ENQUIRIES.

Design of Grain Bins.

X. Y. (Rawalpindi) writes :—"I should be much obliged if you could let me have your opinion on the following points. In designing a group of grain bins to hold a large amount of grain it is found necessary to place a slab of concrete over the whole area under the bins, as the soil cannot take more than one ton on the square foot. It is proposed to make the bins either circular or hexagonal. Each bin will be supported on pillars, and the question is to what extent should the concrete slab under the bins be reinforced, and how can the amount of the reinforcement and the thickness of the slab be arrived at.

"To make matters clearer, I send a drawing showing a plan of a concrete slab required for six circular bins, with the pillars. The pillars here shown are in reinforced concrete. It is probable, however, that the pillars will have to be made in brickwork. If it is found possible to make them of reinforced concrete, would it be advisable to rest them on a continuous circular footing, and should this footing be secured to the concrete slab, and if so, in what way?

"In Ketchum's 'Walls, Bins, and Grain Elevators,' page 192, an almost exactly similar case is given. Here the reinforcement of the base slab is a network of ⅝-in. round steel rods, forming a 9-in. mesh, and the pillars rest on the slab direct, their vertical reinforcements going about one foot into the slab. Ordinarily the slab would be treated as an inverted beam and the reinforcement placed in the upper portion of the slab between the weights. In the present case, however, this cannot be done, and presumably it is sufficient to reinforce the slab as done in the example referred to (cement storage bins)."

We would treat the offsets under pillars as continuous circular beams between pillars, for design of which Professor Ketchum's book may be consulted, and the slab in turn we would treat as circular under the bins, as rectangular between bins, and cantilevered outside. The slab rods we would arrange at right-angles and make allowance for continuity. The calculations should be made in the usual way and are quite simple. It would be impossible for us to give a detailed design as this is somewhat outside our province, and further you do not give sufficient data to enable us to ascertain loads on foundations.

Construction of Storage Tank.

A. B. (Swansea) writes :—"Having in course of construction a storage tank with reinforced concrete, I should like to know the best method of welding the ⅞-in. bars which run through roof of said tank, and if it is possible to weld same with lamp. Also, what length of rods would be most suitable for the following dimensions :—External measurements of concrete are 105 ft. by 59 ft."

Welding should be avoided as far as possible in reinforced concrete. Turnbuckles are preferable for connections of tension rods. We suppose by your sketch that rods are required for beams running the 59-ft. way of the tank. In that case we should have thought it would be better to put intermediate columns so as to reduce the span and economise, when rods could have been lapped over the columns, but if the beams must be 59 ft. long, rods in one length could be obtained from the rolling mills, or, as the full section would not be required to extend throughout, they might be arranged thus—*i.e.*, overlapped at the centre and turned up at the ends to help resist shear. The length of rod we would suggest in that case would be 40 ft. or less.

CORRECTIONS.

Messrs Clay & Sons' New Premises.—In the article on Messrs. R. Clay & Sons' premises, which appeared in our May number, we inadvertently omitted to mention the fact that Mr. Alexander Drew, of Victoria Street, S.W., acted as consulting engineer to this work.

Article on Design of T-Beams in Reinforced Concrete.— We regret that a printing error should have occurred in Mr. R. N. Mirza's article in our June number. On page 440, the fourth line from bottom of page should have read

$$2\left\{\frac{KY^2 - \cdot 5Y(1+K) - {}^2/_3}{2KY - 1}\right\} \text{ by } g,$$

and the fourth equation on page 441 should have been

$$M = c\varphi Bd_s^2.$$

TRADE NOTICES.

The (U.K.) Winget Concrete Machine Co., Ltd., inform us that for the convenience of their customers in London and the Home Counties, and to cope with their increasing business, they have opened a London office at 5, East India Avenue, London, E.C., in charge of Mr. Victor Kemp, who has been appointed manager.

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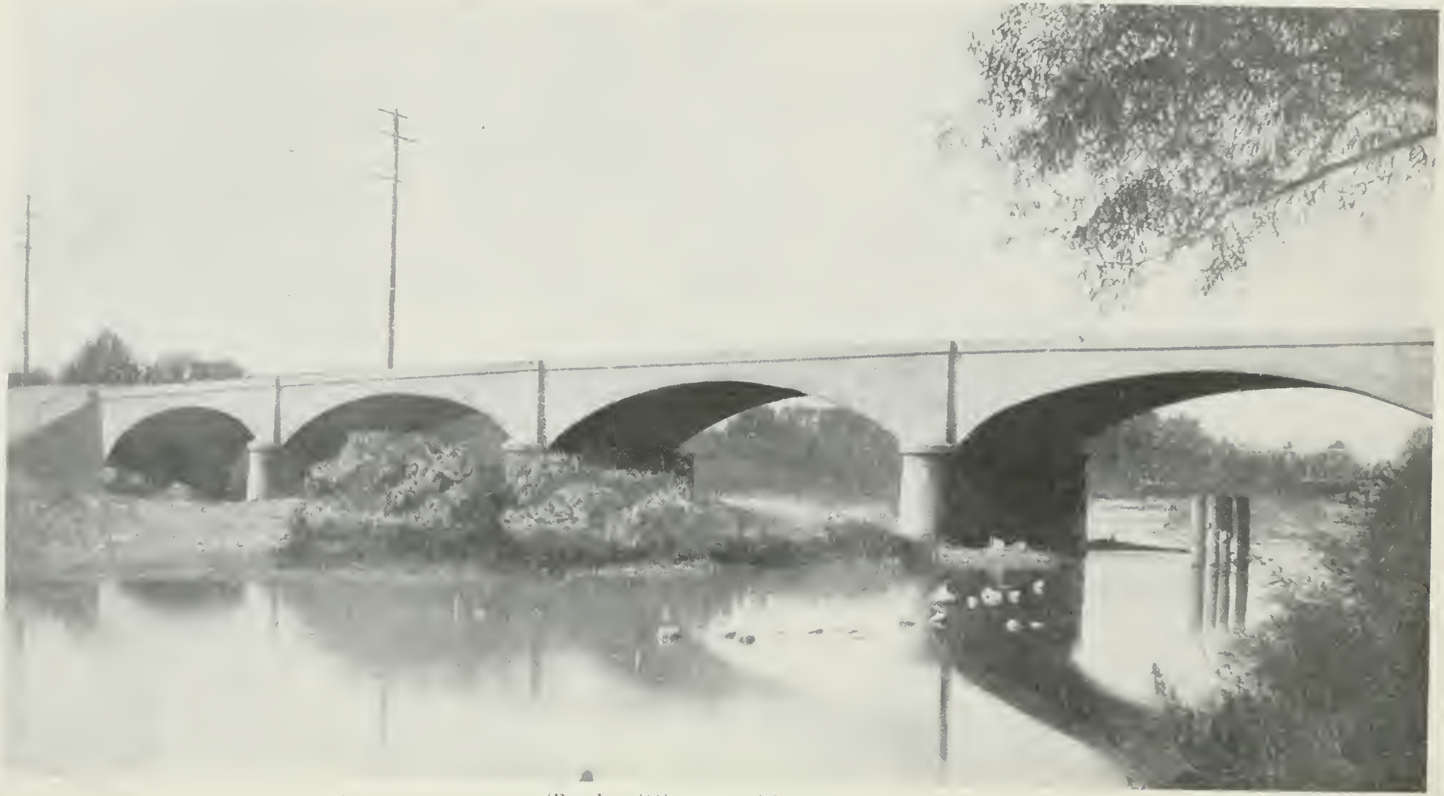
Face p. 559.



(For description see article on p. 606.)

Reinforced Concrete Bridge at Plainfield, Indiana.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 8. LONDON, AUGUST, 1912.

EDITORIAL NOTES.

REINFORCED CONCRETE AND FIRE PREVENTION.

THAT the application of reinforced concrete, when properly designed, of suitable composition and carefully executed with due regard to fire hazard, affords one of the best means of obtaining a fire-resisting building can scarcely be disputed, and we hold that the earlier uses of the new material for structural purposes in this country was largely due to the influential advocacy on its behalf of those primarily concerned in fire prevention.

It thus behoves both designers and contractors engaged on reinforced concrete to be most careful not to take undue risks where fire hazard may be expected. The unfortunate tendency to scamp in the covering of the metal members should be most strictly condemned, as also the use of unsuitable aggregates, for any weakness from the fire point of view must necessarily first make itself felt where the protective covering is insufficient or liable to rapid disintegration. There have been failures abroad of reinforced concrete structures, or portions thereof, due entirely and solely to scamping or unsuitable aggregate as distinct from any fault in the design *per se,* and it would be regrettable indeed if any similar record was obtained at fires in these islands.

THE PROTECTIVE COVERING OF THE METAL.

Unfortunately the extent of the covering for the metal members in reinforced concrete construction is dealt with very much by rule-of-thumb, based, it is true, largely upon the results of the fire tests of the British Fire Prevention Committee, but the latter do not as yet afford data of sufficient detail to enable precise deductions being arrived at. The committee, it is true, has tested quite a number of concrete and reinforced concrete floors, and certain deductions can be made of the relative resistance of the different systems of construction that have been under investigation, but the research work done to date by no means fully covers the subject, for there is still lacking the necessary information as to the extent of the covering essential for different sections, thicknesses and qualities of the metallic reinforcement, given identical aggregates and identical conditions.

A common rule is to provide a protective covering of from 2 in. to 2½ in. depth in main girders. For columns the protection is expected to be greater; for small girders and small panels it is less, and the practicable minimum of protection under a panel is supposed to be 1 in. In such panels it is generally assumed that the rods are of light section, and that the concrete aggregate will pass through a sieve of ½ in. mesh. As a matter of fact, this minimum of 1 in. is on the dangerous side, and 1½ in. is more reliable for any horizontal panel of 7 to 10 ft. span. Such a protective covering of 1½ in., however, adds to the weight of a floor, adds to its thickness, and has various other disadvantages which economical designers chafe at, and the result has been that every possible excuse is invented to lessen the thickness of the protective covering in question, and sometimes the ordinary plastering applied to the soffit of a floor panel is reckoned in as part of the thickness of 1 in. asked for.

Now of the many excuses for a thin covering that come before us, probably the most sensible is the one that is based on the argument that, if the reinforcement be kept to the thinnest possible section, the covering may be materially reduced, owing to the lesser effect of conductivity to the smaller sections of metal than to those of greater scantling. In other words, the argument that the thinner the reinforcement rods the thinner may be the covering seems a feasible one, although the point has not as yet been scientifically proved.

FURTHER INVESTIGATION ADVISABLE.

To obtain a reliable solution we would welcome a systematic test of floor panels of identical span and identical thickness reinforced with different forms of reinforcement, from the thinnest mesh to the common forms of round bars, the conductivity of the concrete, and the temperature of the metal to be carefully recorded. If such investigations were conducted, and the reinforcing metal protected with covering material of, say, $\frac{3}{4}$ in., 1 in., $1\frac{1}{2}$ in., and 2 in. pyrometer points systematically applied within the concrete, the whole problem could be solved once and for all, much as the question of the fire resistance of different concrete aggregates was solved by the British Fire Prevention Committee some years back, when the Associated Portland Cement Manufacturers, in a most public-spirited way, arranged for a careful investigation of the matter then at issue.

A step in the direction of obtaining further comparative data has already been taken, we are glad to say, by the Committee in undertaking a test with a floor having a modern triangular mesh reinforcement, the test being conducted on identical lines with former tests with the earlier types of reinforced concrete floors, both in respect to fire and load requirements, application of water under pressure, etc. Fortunately, these particular tests will also be undertaken upon spans practically identical with those of the earlier floors tested, so that scientific comparisons can be made to a certain extent. These tests are due as we go to press, so that there will be no great delay in obtaining some authentic record.

DIFFERENT REINFORCEMENT TO MEET DIFFERENT CIRCUMSTANCES.

There is a tendency, which should be combated, to suggest that all reinforced concrete systems have an equal value and fire resistance, which is, of course, not the case. That the "Visintini" floor failed owing mainly to an entirely insufficient protective covering when tested at the Committee's testing station will be remembered by many, also that the Coignet floors of two different forms showed highly satisfactory, but quite distinctive, results is still borne in mind by those concerned. The tests with the expanded metal and the indented bar floors both told most useful tales, whilst, to mention another piece of research of far-reaching effect, the armoured tubular floor test taught architects the value of an air cushion for certain classes of construction. A few more such records may eventually clearly show the precise advantages of one or more systems of reinforcement for the specific purpose of serving as partial fire retardents, whilst some other system or systems may be definitely found almost impregnable to the fiercest conflagration conditions. Given the relative fire resistance to meet different conditions, various systems of reinforcement can then be specified for different classes of structure, and economy will result. The impregnable floor is not wanted for a block of ordinary city offices, but it is very much in place in a cotton warehouse. Given greater discrimination, building costs will accord better with the requirements of the structures to which reinforced concrete is applied, and reinforced concrete contractors will have an increased sphere of activity.

Thus we advocate further investigation into the fire aspects of reinforced concrete, combined with the greatest possible care in design and execution. Investigation will mean a general cheapening of its use for the more ordinary forms of building and a more suitable application of the material generally, whilst care should prevent such accidents as might mean a set-back and cause unjust suspicion against the fire-resisting qualities of reinforced concrete.

There is the very great probability of reinforced concrete becoming the primary building material of the future wherever the question of fire hazard pertains, and everything should be done to advance its chances in this direction and to prevent the slightest possibility of this position being questioned.

REINFORCED CONCRETE AND LOCAL AUTHORITIES.

We have already called attention to the fact that local authorities are slow in making alterations in their building by-laws so as to admit of the use of reinforced concrete in every part of the building, and we publish a letter from Mr. Ridley, which comments on the attitude of the West Ham Corporation, who have stated that they intend to alter their by-laws, but not to provide in such revision for the full use of reinforced concrete.

Apparently this local authority does not understand the illogical attitude they have taken up, viz., that while reinforced concrete can be used for the whole of the interior of a building, for the foundations, columns, beams, floors, etc.—which can be constructed without reference to any by-laws—the outside walls and foundations thereto, which are often, from a structural point of view, of minor importance, cannot be constructed in reinforced concrete, owing to the existing by-laws or the proposed revisions to same.

We admit we are a very conservative nation in the matter of accepting new methods or ideas, but the value of reinforced concrete construction has been well proved, and several important municipal and other authorities have acknowledged this by providing for its use to the fullest extent, revising and framing their by-laws for the purpose, and in some cases its use is permitted and by-laws ignored. If other public bodies follow the unfortunate policy of the West Ham Corporation it will greatly retard the scientific and economical progress of building construction and further handicap the British manufacturer, as other nations, recognising the economical value of reinforced concrete structures—as compared to the ordinary method—have encouraged its use, thereby permitting the erection of more durable and less costly buildings, saving capital outlay in this direction, and thereby allowing the use of such saved capital for the furthering of commercial interests in other directions.

The Government—who largely employ this form of construction—have long recognised the utility of reinforced concrete and the saving its employment effects. The saving in cost varies from 15 to 25 per cent., and the saving in maintenance is also substantial.

The London County Council have framed new by-laws especially for this purpose (see our issue of December, 1911), and this should alone convince, but might we suggest that local authorities, where in doubt as to the suitability and efficiency of this form of construction, should call upon the Concrete Institute or some similar recognised body concerned in this class of work to send delegates to explain its advantages and present facts as to its adoption elsewhere.

The following is the letter referred to above :—

Reinforced Concrete Construction in West Ham.

Sir,—With reference to the discussion of the West Ham Town Council relating to the matter of building by-laws as applied to reinforced concrete construction, I feel bound to comment on the attitude adopted, both in the interest of the borough and because I feel such an attitude will greatly retard the progress of scientific and economical building construction. I would preface my remarks by stating that I wish in no way to comment upon the Council's decision regarding the plans for the women's settlement, as, although interested as consulting engineer for the structure, I follow the difficulty in accepting the plans, considering the present existing by-laws. I understand that the revision of the by-laws is now being considered, but (according to the Press report) the Council do not contemplate inserting any clause or making any provision whereby it will be permissible for buildings of entirely reinforced concrete construction to be erected.

This, in my opinion, especially in the case of the West Ham district (where the sub-soil is very unstable), is greatly against the interest of all concerned. Under the existing by-laws the employment of reinforced concrete construction is permitted, and is largely used, for all parts of a structure excepting the external walls and the footings supporting same. Therefore any attitude whereby the by-laws are not now framed to permit the construction of such walls and footings in the manner proposed is illogical, and the use of reinforced concrete for such work is of the greatest advantage. As compared with the general form of construction, reinforced concrete has the following advantages : (1) Homogeneous construction. (2) The economical employment of concrete and steel, whereby great strength is obtained in small areas, thus reducing the entire weight of the building, and by so doing decreasing the loads received by the ground. (3) That buildings so formed are fire resisting and less liable to damage by fire than any other form of construction. (4) Saving in capital outlay. (5) Minimum of maintenance.

Although the use of reinforced concrete has in this country not been employed to its fullest extent, its utility is receiving greater recognition year by year, and many leading local authorities have amended their by-laws in order to admit of its employment in its entirety, and the London County Council have obtained the sanction of Parliament to special by-laws whereby its use is fully permitted. These by-laws were framed with the assistance of all the leading institutions and experts, and under its provisions external walls may be formed 4 in. thick, and the footings also, all in reinforced concrete. These by-laws are now waiting the final sanction of the Local Government Board, but the London County Council have given and are giving their permission for the erection of entire reinforced concrete structures, prior to the final sanction of such Act. The Government are largely using this form of construction, and the Local Government Board have advised the erection of schools in this material, having made an order which renders void any local by-laws which might restrict the use of reinforced concrete in this respect. Not only is reinforced concrete being employed for ordinary structures, but some of the more important engineering works, such as bridges, piers, wharfs, docks, etc., are being formed in this material.

I noticed it was stated that if this form of construction were permitted in the borough there would be great risk of bad workmanship and inferior materials, and therefore unsafe buildings. I would point out that this might also occur with the ordinary type of construction, and that a good contractor in the usual form of construction would also perform good work with reinforced concrete construction. I might also mention that bricks of inferior quality can be used, and, providing that the walls are constructed of the thickness required by the by-laws, the Council have no power to prevent the use of such inferior bricks. The Council have officials who should be competent to see that the work is carried out in a proper manner, so far as the Council are concerned. I do not, under any circumstances, presume to criticise the ability of the officials, as there is no doubt they must be conversant with the reinforced concrete form of construction as it is now used in the borough for floors, beams, columns, piers, etc.

M. Noel Ridley, M.Inst.C.E., M.C.I.

1, Mitre Court, Temple, E.C.,
July 11th, 1912.

LECTURE HALL, YORK MUSEUM, BUILT OF REINFORCED CONCRETE

A very interesting piece of reinforced concrete work has recently been carried out in the erection of the Lecture Hall at York Museum. This was built on the site of an old Abbey, some of the ruins of which were incorporated in the new structure. Details of the work are given below.—ED.

This hall has been erected on the site of St. Mary's Abbey, and forms an addition to the Yorkshire Philosophical Society's Museum at York; and it is interesting to note that the ruins of the Abbey were not disturbed, but where necessary were incorporated in the new work, and this to a certain extent influenced the foundation work during execution.

The building was designed by Mr. E. Ridsdale Tate, architect, of York, and is constructed entirely of reinforced concrete on the Kahn system, designed by the Trussed Concrete Steel Co., Ltd., of Westminster.

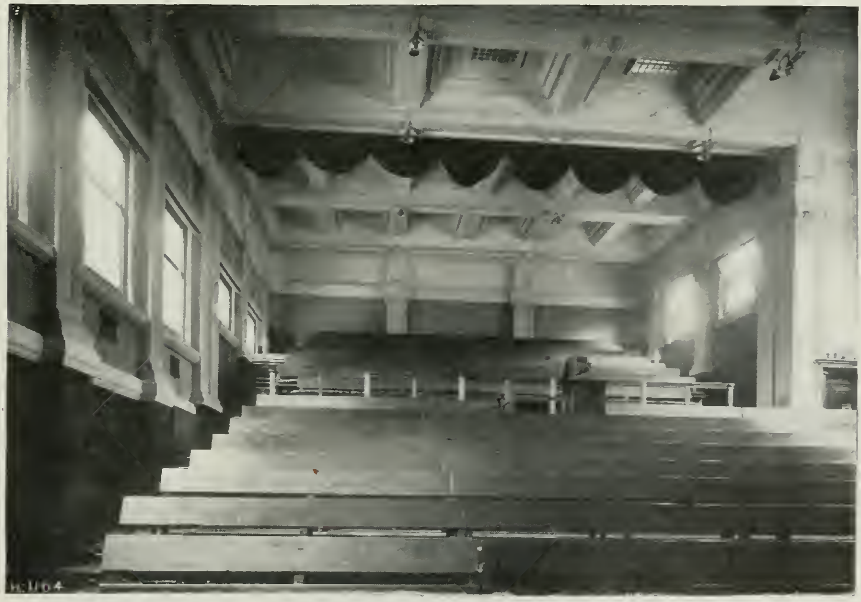


Fig. 1. Interior View of Hall.

Lecture Hall, York Museum.

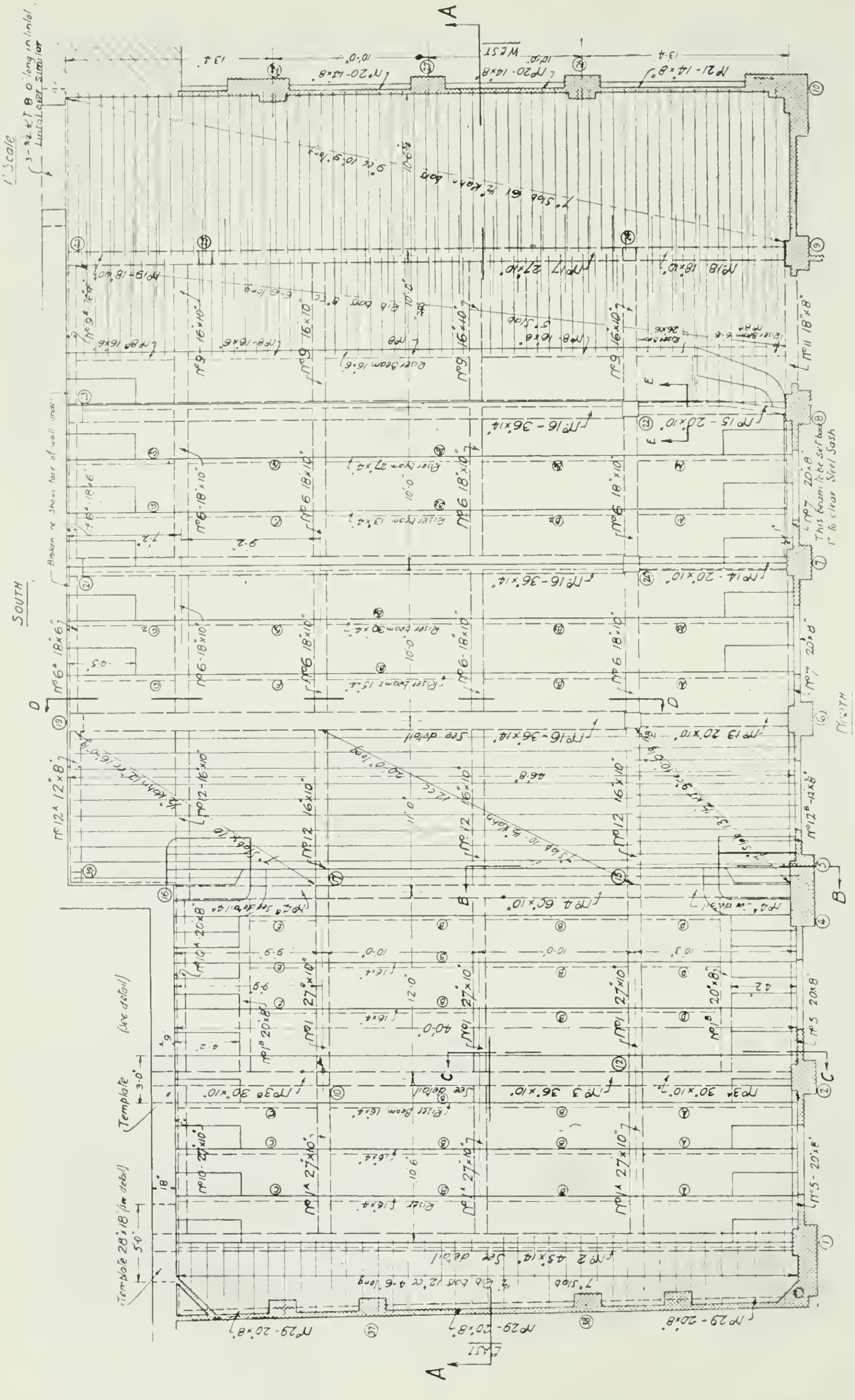


Fig. 2. Plan Showing Arrangement of Beams, etc.
Lecture Hall, York Museum.

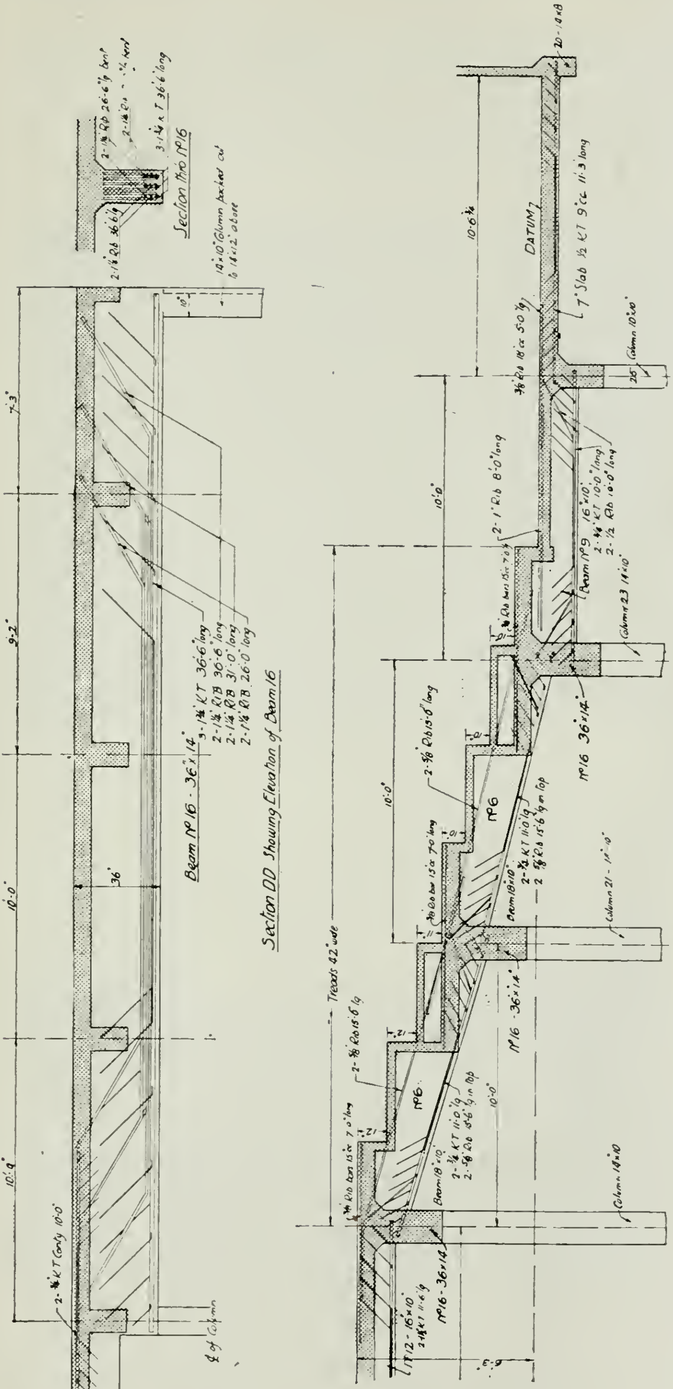


Fig. 3. Section Showing Construction of Lower Tiers.
Lecture Hall, York Museum.

One interesting feature is that all the cornices and other mouldings were cast at the time of the construction, and the whole of the exposed surfaces, including the walls, are just as they left the centering, with the exception that they have been cleaned down. This entailed great care in putting together the wooden forms, but saved the necessity of rendering or other surface treatment, and guaranteed a permanent finish. The whole of the enrichments were also cast *in situ*, and the results are remarkably good.

The foundations are carried on a bed of clay which exists about 5 ft. below the surface, and as some of the ruined walls of the Abbey were existing at certain points they were utilised to carry portions of the new work after careful investigations and calculations had been made, and the bearing capacity of the soil had been ascertained. A great deal of trouble and time were expended in these preliminary investigations to prevent the possibility of any future unequal settlement, and the result has been highly successful.

The new building consists of a lecture hall about 78 ft. long and 47 ft. wide, with a height of 31 ft. 6 in. from the lowest level of the hall floor, and the general arrangement of the beams, etc., is shown on the plan illustrated in *Fig.* 2. The purpose of the hall necessitated the floor being constructed in tiers, which rise up to a height of about 18 ft. 6 in., as shown in the sections indicated in *Figs.* 3 and 4, and this greatly influenced the design. These tiers were formed with a rise of 10 to 12 in. and a width of either 3 ft. or 3 ft. 6 in., according to the position, and were carried on stringer beams placed at about 10-ft. centres Generally speaking, the riser beams were constructed with a thickness of 4 in. and reinforced with one ¾-in. Kahn bar in the lower surface and one ½-in. continuous Rib bar in the upper surface, while the tread was formed as a slab 3 in. thick, reinforced with 5/16-in. diameter rods at 7-in.

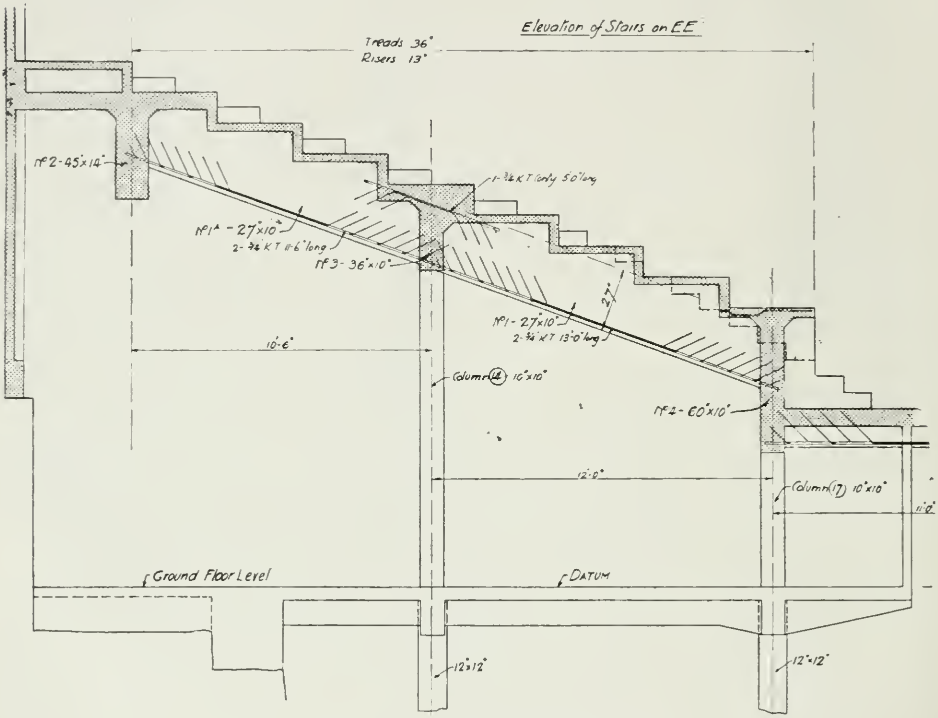


Fig. 4. Section Showing Construction of Highest Tiers.
Lecture Hall, York Museum.

centres. The stringer beams were supported on the main beams, which were placed at distances of 10 ft. to 12 ft. apart, and the size of these stringer beams varied from 27 in. by 10 in. to 18 in. by 10 in., the former of these being reinforced with two ¾-in. Kahn bars in the lower surface.

A series of tests were conducted by Mr. F. W. Spurr, the City Engineer, on behalf of the York Corporation, and in every case the load applied was 1½ times that which the member was calculated to carry. The photograph in *Fig.* 5 shows the test on one of these stringer beams when the load equalled 11¼ tons over an area of 8 ft. 10 in. by 10 ft., and although the permissible deflection was ·294 in. the actual deflection under the test load was only ·027 in.; and such a result must be considered highly satisfactory.

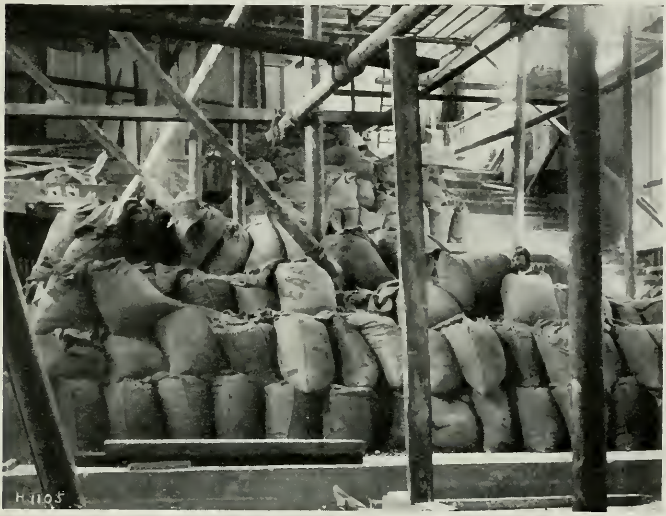


Fig. 5. Test on Floor Beam and Raking Stringer.

The beams carrying the stringers had large spans owing to the fact that only one row of columns was provided under the main body of the hall, and this was placed at a distance of 10 ft. from the north wall, thus leaving a distance of 35 ft. as the clear span from such columns to the south wall. A typical beam is that illustrated in *Fig.* 2, this carrying the ends of three stringer beams on either side and having

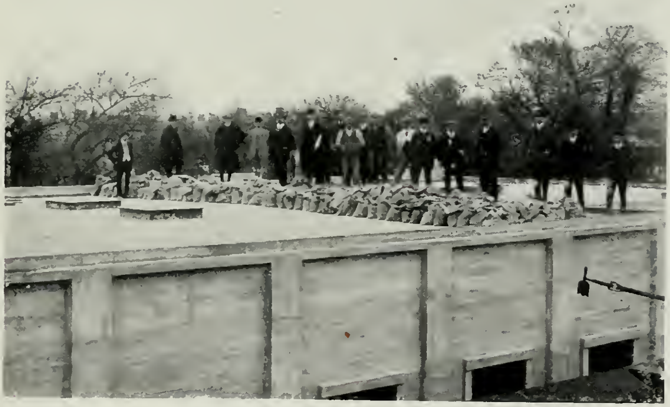


Fig. 6. Test on Main Roof Beam.
Lecture Hall, York Museum.

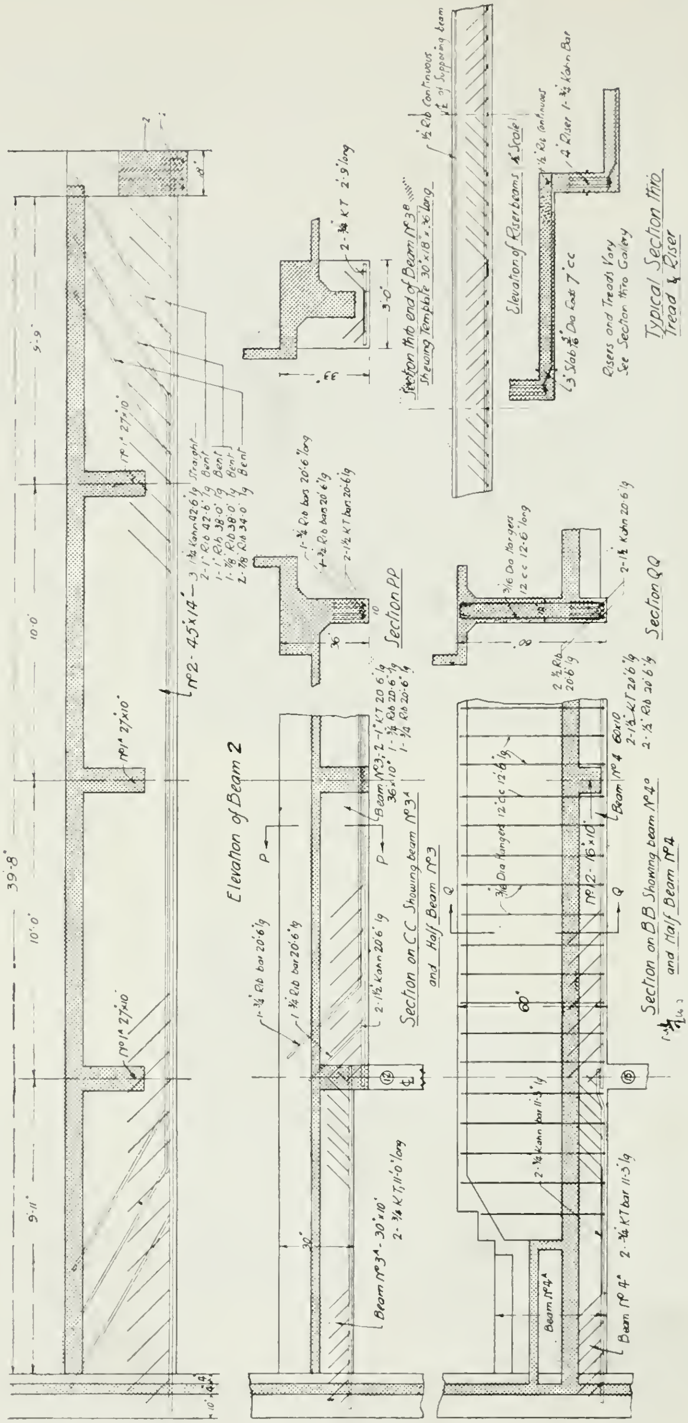

Elevation of Beam 2
Nº2 - 45"x14"
Nº1ᴬ 27"x10"
39'-8"
9'-11"
10'-0"
10'-0"
9'-9"
Section PP
Section QQ
Section on CC Showing beam Nº3ᴬ and Half Beam Nº3
Section on BB Showing beam Nº4ᴬ and Half Beam Nº4
Beam Nº3ᴬ - 30"x10" 2-¾ KT, 11'-0" long
Beam Nº4ᴬ
Beam Nº4ᶜ 2-¾ KT bar 11'-3" lg
2-¾ Kahn bar 11'-3" lg
3/16 Dia Hangers 12" cc 12'-6" lg
Nº2 - 16"x10"
Beam Nº4 60x10
60"
Section thro end of Beam Nº3ᴮ Shewing Template 30"x18"x36" long
Elevation of Riser beams
Typical Section thro Tread & Riser
Risers and Treads Vary See Section thro Gallery
3" Slab 5/16" Dia Rods 7" cc
4" Riser 1-¾ Kahn Bar

Fig. 7. Details of Beams and Treads.
Lecture Hall, York Museum.

an over-all size of 36 in. by 14 in. The reinforcement consisted of three $1\frac{3}{4}$-in. Kahn bars and six $1\frac{1}{4}$-in. Rib bars, all placed in the tension area; and the test on this beam, which is also illustrated in *Fig.* 5, consisted of loading a floor area of 35 ft. by 10 ft. with $40\frac{1}{2}$ tons of ballast, such load producing a deflection of only ·047 in., whereas the permissible deflection was 1·166 in. The lowest portion of the floor, which is horizontal for a distance of about 16 ft. from the west wall, is formed with 7 in. of concrete, reinforced with $\frac{1}{2}$-in. Kahn bars at 9-in. centres for 10 ft. of the distance, and the eastern edge is supported by three beams, one of which has a span of 26 ft. 9 in. and a size of 27 in. by 10 in., reinforced with two $1\frac{3}{4}$-in. Kahn bars and five 1-in. Rib bars in the lower surface and two $1\frac{3}{4}$-in. Kahn bars and four 1-in. Rib bars in the upper surface. This beam was tested by loading a floor area of 26 ft. 9 in. by 10 ft. 3 in. with 32 tons of ballast in bags, giving a load equal to $2\frac{1}{4}$ cwt. per sq. ft., when the greatest deflection was only ·066 of an inch, whereas the permissible amount was ·890 of an inch.

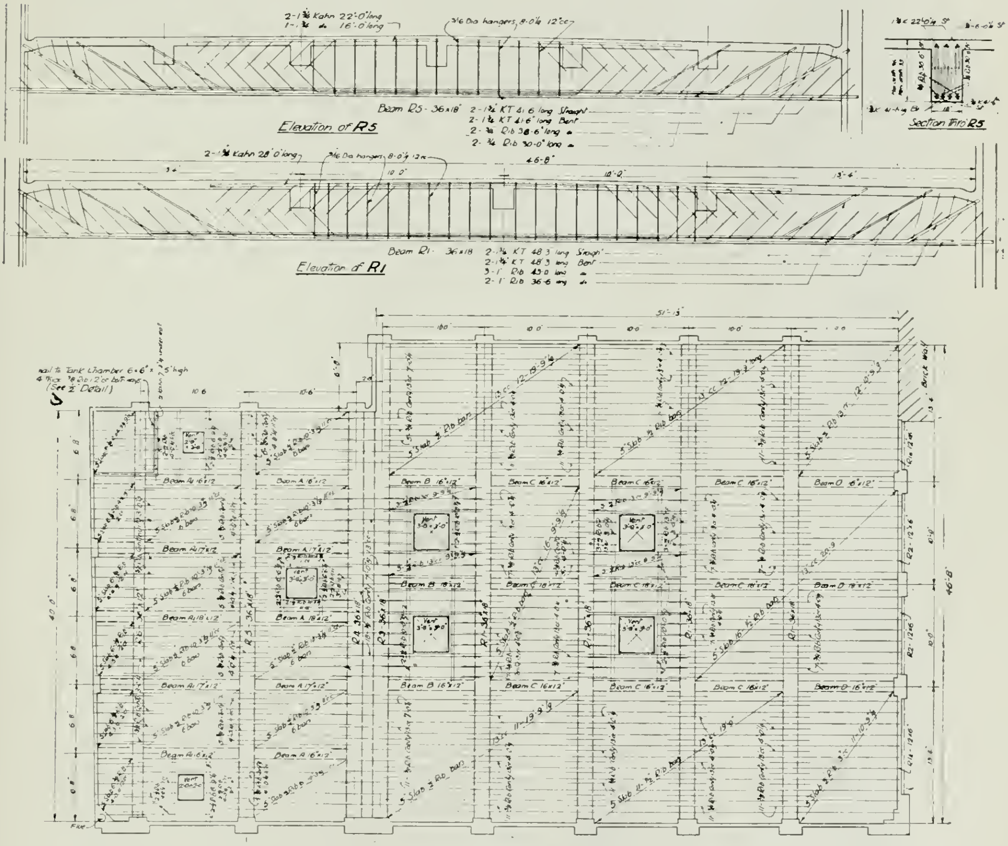
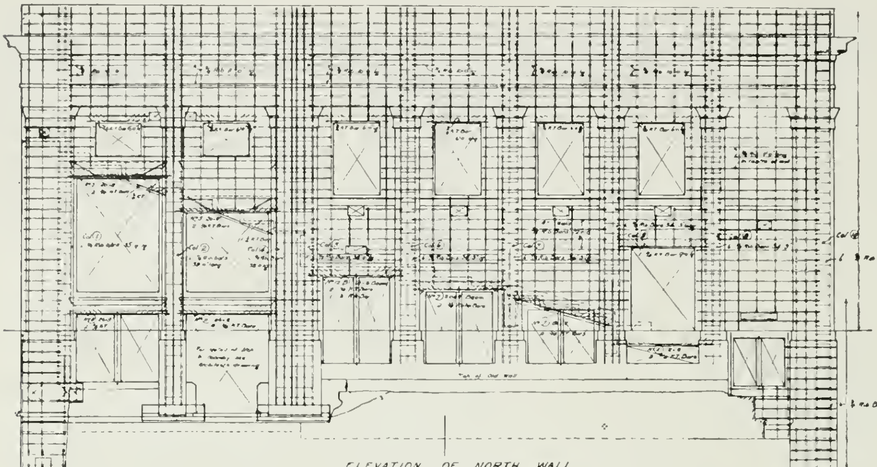


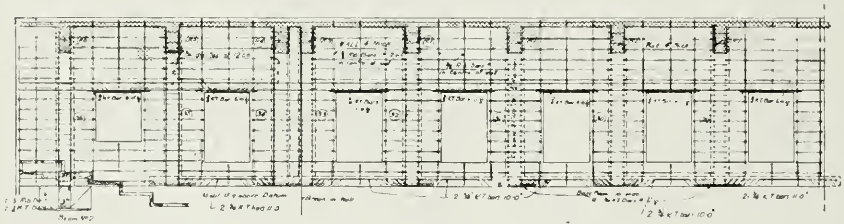
Fig. 8. Roof Plan and Details.
Lecture Hall, York Museum.

Other typical floor details are those illustrated in *Fig.* 7, which shows a section of the tread and riser and a large beam having a span of 39 ft. 8 in., which occurs at the highest level of the floor. This beam carries the ends of three stringer beams and one edge of the 7-in. slab which forms the floor between the beam and the east wall, and it has a depth of 45 in. and a width of 14 in., with reinforcement on the lower surface of three $1\frac{3}{4}$-in. Kahn bars, three 1-in. Rib bars, and three $\frac{7}{8}$-in. Rib bars. The northern end of this beam is carried on a column 21 in. by 18 in., reinforced with six $\frac{7}{8}$-in. Rib bars and $\frac{3}{16}$-in. wire ties at 12-in. centres, which is carried down to the top of an existing masonry wall, which is about 5 ft. 6 in. below the ground-floor level, in such a manner that part of the column comes on the wall and the remainder is carried on a reinforced pier 4 ft. by 18 in., constructed against the wall, and carried down a further distance of 9 ft., where it is carried on a foundation 8 ft. by 4 ft. 2 in., well reinforced; the southern end of the beam being supported on an existing wall with a reinforced concrete template 28 in. deep, 5 ft. long, and carried 18 in. into the wall, with two $\frac{3}{4}$-in. Kahn bars and two $\frac{1}{2}$-in. Rib bars as reinforcement. This example should serve as an illustration of the



ELEVATION OF NORTH WALL

For thickness of wall & exact size & position of openings work to Architect's drawings



ELEVATION OF SOUTH WALL

For exact size & position of openings work to Architect's drawings

Fig. 9. Details of Wall Construction.
Lecture Hall, York Museum.

variation of design that was required to adapt the new work to its position, giving a great deal of interest not ordinarily met with.

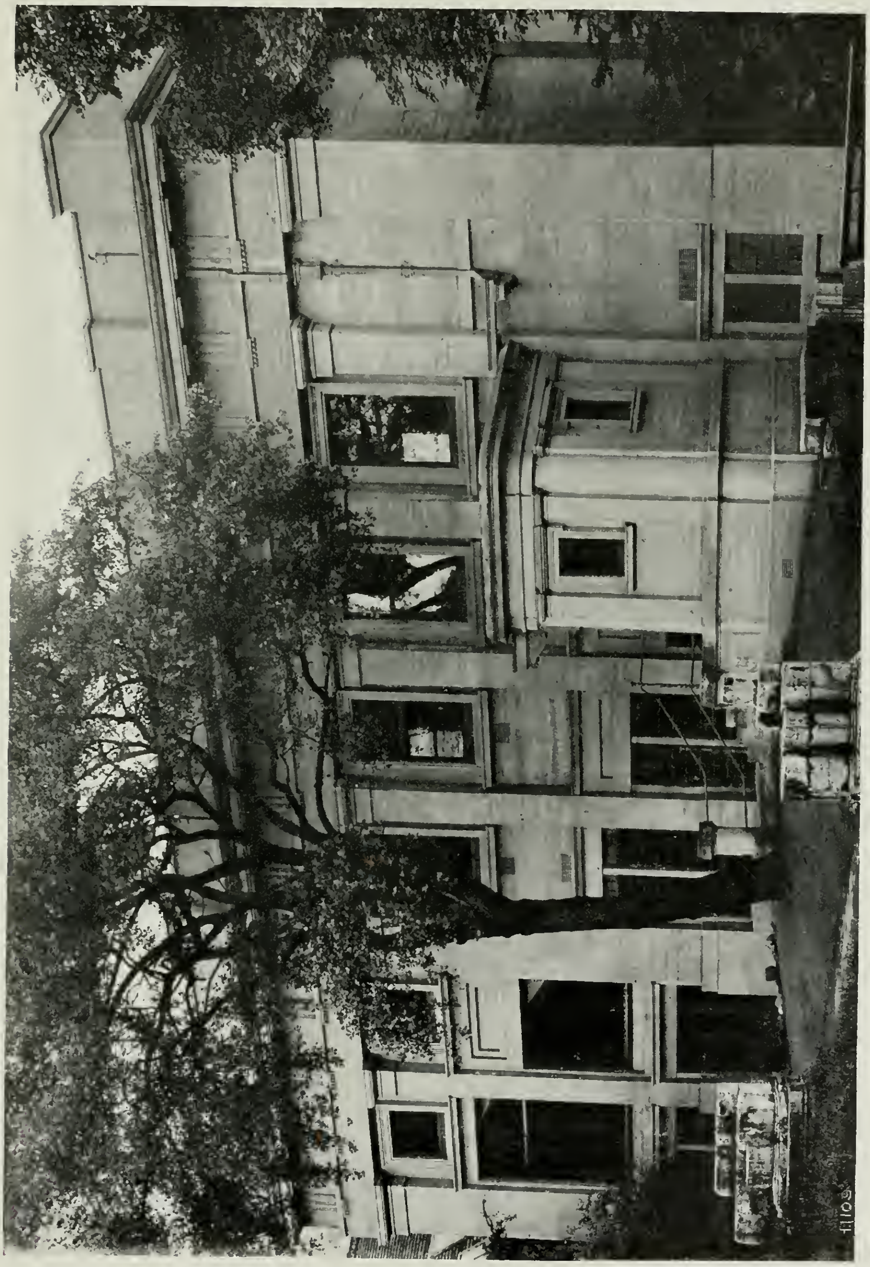


Fig. 10. Exterior View of Finished Building.
LECTURE HALL, YORK MUSEUM.

The plan of the roof is illustrated in *Fig.* 8, and it will be seen that the whole of this is constructed of reinforced concrete, being formed with main beams, secondary beams, and slabs. The secondary beams, however, are merely false beams, 12 in. wide, reinforced with one ¾-in. Kahn bar, introduced for the purpose of effect on the ceiling, as illustrated in the photograph of the interior, *Fig.* 1.

The main beams are interesting, as they have a clear span of 46 ft. 8 in., claimed to be the largest span of any flat beams in this country, and the size is 36 in. by 18 in., with reinforcement in the tension area of four 1¾-in. Kahn bars, and five 1-in. Rib. bars, while two 1¾-in. Kahn bars are introduced in the compression area for a distance of 14 ft. on either side of the centre of the span, the two sets being connected together with 3/16-in. diameter links spaced at 12-in. centres. These beams carried a roof area of 46 ft. 8 in. by 10 ft., and a test was conducted, as illustrated on *Fig.* 6, by loading this area with 12½ tons of ballast, being 60 lb. to the square foot. The deflection at the centre of the beam under this load was only ·043 in., whereas the permissible deflection was 1·555 in., such a result being eminently satisfactory. The roof slabs, carried by these beams, have a span of about 10 ft., and they were constructed of concrete 5 in. thick reinforced with ½-in. Rib bars at 13-in. centres, with ⅜-in. Rib bars to preserve continuity, 4 ft. long, placed at 13-in. centres over the main beams.

Several openings for ventilation purposes were required in the slabs, and additional reinforcement was provided around these. A small tank chamber was also formed in reinforced concrete in one corner of the roof, and this was 6 ft. square, inside sizes, and 5 ft. deep, with 4-in. walls and ⅜-in. Rib bars vertically and horizontally.

The type of construction employed for the walls is illustrated in *Fig.* 9, and it will be seen that wall columns were carried down under the main beams, and the panels between were formed with 4 in. of concrete, reinforced generally with ⅜-in. Rib bars horizontally and vertically. The south wall was constructed upon an existing wall, the top of which was about 15 ft. above the ground-floor level, and a base beam 10 in. wide was formed to distribute the weight over same. A workroom floor was constructed under a portion of the hall, where the height under the tiers permitted this, and consequently some of the internal columns were extended through this.

A typical column section was 10 in. square, reinforced with four ½-in. Rib bars and 3/16-in. diameter binding at 9-in. centres, increased to 12 in. square where supporting workroom floor, reinforced with four 1-in. Rib bars and similar binding.

The wall columns varied according to their position, but, generally speaking, they were reinforced with six ⅞-in. Rib bars and 3/16-in. diameter links spaced at 12-in. centres. There were really three types employed—viz., single wall columns, twin columns, and angle columns, and the size and position were governed by the architect's drawings.

The photograph illustrated in *Fig.* 10 shows the finished building on the north side, and it will be seen that a porch was constructed here, this coming over the ruined wall of the south transept of the abbey. The porch was formed in reinforced concrete, together with the steps leading up to the floor level. In this view some of the remains of the abbey can be seen, and it is symbolical of the progress of modern times that the old masonry building erected many years ago in the Gothic style should be replaced by a Renaissance building constructed of reinforced concrete and designed on highly scientific lines.

The building throughout is admirably designed and constructed, and the City Engineer reported that the tests were very satisfactory, thus reflecting great credit on all concerned. The contractors for the work were Messrs. J. & T. Biscomb & Sons, of York.

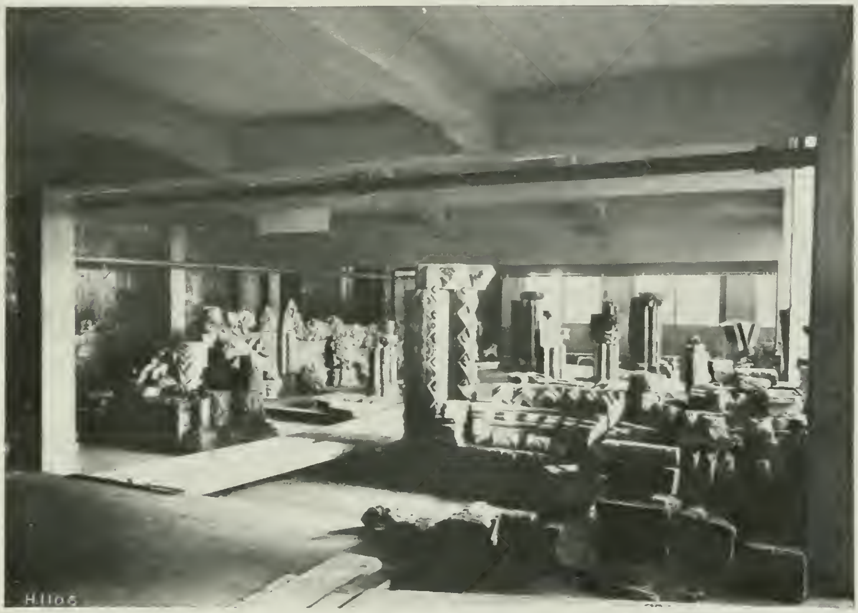


Fig. 11. Lower Hall and Architectural Museum.
Lecture Hall, York Museum.

BENDING MOMENTS IN BEAMS WITH PARTLY FIXED ENDS

AND THE BENDING MOMENTS THEY PRODUCE IN THEIR SUPPORTS.

By H. KRUSE, M.Inst.C.E. Copenhagen.

The following article on the Bending Moments of Beams will be interesting, in view of Mr. Béhar's lecture before the Concrete Institute published in our July issue. We may say that the present contribution was received by us some time before Mr. Béhar's paper was read.—ED.

THE question of figuring out the exact bending moments in beams with partly fixed ends or in continuous beams is, with a few exceptions, receiving little or no consideration from designers of reinforced concrete structures, although this method of construction offers many opportunities where such calculations would be of economical value and where they are absolutely necessary in order to know the strength of the structure.

If we take the case of a beam simply supported at the ends and uniformly loaded with W, we know that the bending moment at the centre of the beam is $\frac{Wl}{8}$ and the moments at the ends of the beam are 0.

If the beam is fully fixed at the ends the bending moments are $\frac{Wl}{24}$ at the centre, and $\frac{Wl}{12}$ at the ends. It will be seen that in a beam partly fixed at the ends the bending moments may vary from $\frac{Wl}{8}$ to $\frac{Wl}{24}$ at the centre and from 0 to $\frac{Wl}{12}$ at the ends according to the degree of fixedness, and this variation is so large that it deserves some consideration.

In the following remarks the writer endeavours to solve some of the most ordinary cases without the use of infinitesimal calculus, and the method used is based on the following maxim:

> The angular deflection, or the angle a deflected beam forms with its original horizontal axis, is found as the shearing force in an imaginary beam of same span, simply supported at the ends, and loaded with $\frac{M}{EI}$, M being the bending moment diagram for the original beam, I its moment of inertia, and E the modulus of elasticity of the material.

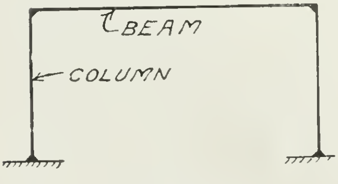


FIG. 1.

1st Case (see *Fig.* 1).—A horizontal beam is supported at each end by a column, to which it is rigidly connected. Considering the column first by itself, it may be taken as a beam fixed at the lower end B, where it is connected to the foundation, and acted upon at the upper end A by a bending moment M_a, produced by the beam (see *Fig.* 2).

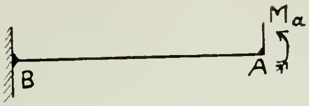


FIG 2

The bending moment M_a acting on AB produces a reacting bending moment M_b at the end B, where it is fixed, and the bending moment diagram will therefore be represented by the area $BACD$ (see *Fig.* 3), and we now consider the imaginary beam AB loaded with this area.

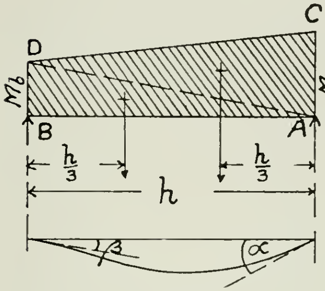


FIG. 3.

The shearing force at B in the imaginary beam is the reaction B from the area $BACD$.

We divide $BACD$ into the two triangles, BAD and ADC.

Area of $\triangle BAD$ is $\frac{hM_b}{2}$, and the distance from its centre of gravity to A is $\frac{2h}{3}$. Area of $\triangle ADC$ is $\frac{hM_a}{2}$, and the distance from its centre of gravity to A is $\frac{h}{3}$.

By taking the moment about A we get:

$$Bh = \frac{hM_b}{2}\,\frac{2h}{3} + \frac{hM_a}{2}\,\frac{h}{3} = \frac{M_b h^2}{3} + \frac{M_a h^2}{6} = \frac{h^2(2M_b + M_a)}{6}$$

$$\text{or } B = \frac{h(2M_b + M_a)}{6}$$

The angular deflection β of the column at B is then:

$$\beta = \frac{B}{EI_c} = \frac{h(2M_b + M_a)}{6EI_c} \quad . \quad . \quad . \quad . \qquad (1)$$

where I_c is the moment of inertia of the column.

As the column for which we want to find the angular deflection is fixed at B we must have $\beta = O$, and consequently: $2M_b + M_a = O$ or $M_b = -\frac{M_a}{2}$, which means that the moment produced at the lower fixed end of the column is acting opposite to and half as big as M_a. By taking the moment about B we get in the same way as before:

$$Ah = \frac{hM_b}{2}\,\frac{h}{3} + \frac{hM_a}{2}\,\frac{2h}{3} = \frac{h^2(M_b + 2M_a)}{6}$$

$$\text{or } A = \frac{h(M_b + 2M_a)}{6} \text{ and as } M_b = -\frac{M_a}{2}$$

$$A = \frac{h\left(-\frac{M_a}{2} + 2M_a\right)}{6} = \frac{h\,M_a}{4}$$

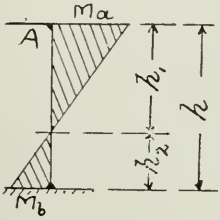


FIG: 4

The angular deflection at A will then be:

$$\alpha = \frac{A}{EI_c} = \frac{h\,M_a}{4EI_c} \quad . \quad . \quad . \quad . \qquad (2)$$

In *Fig.* 4 is shown the bending moment diagram for the column AB. From the *Fig.* we see:

$$\frac{h_1}{h_2}=\frac{M_a}{M_b}=2 \text{ or } h_1=\frac{2h}{3} \text{ and } h_1=\frac{h}{3}$$

and it is thus seen that the point of contraflexure in the column is situated at a distance of one-third of its height from the lower end.

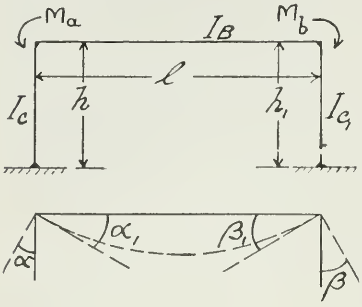


FIG. 5

We will now have to consider the beam supported by the columns (see *Fig.* 5). The load on the beam is the uniformly distributed load W.

The bending moment on the beam, if simply supported at the ends, would be represented by the parabola AGB (see *Fig.* 6) where $HG=\frac{Wl}{8}$, but on account of the ends of the beam being partly fixed, the end moments M_a and M_b are produced, and the resulting bending moment diagram will be as shown by the shaded area in *Fig.* 6.

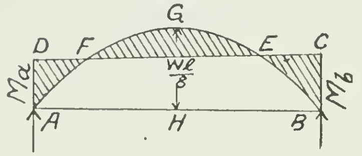


FIG. 6.

The angular deflection α_1 of the beam AB at A we get by considering an imaginary beam loaded with the bending moment diagram. The shearing force at A in the imaginary beam AB is the reaction A from the bending moment diagram.

The area of $AGBHA$ is $\frac{Wl}{8}\,\frac{2l}{3}=\frac{Wl^2}{12}$, and the reaction from this at A is $A_1=\frac{Wl^2}{24}$, and putting $\frac{Wl}{8}=M$,

$$A_1=\frac{Ml}{3}.$$

The area of $ABCD$ is $\frac{l(M_a+M_b)}{2}$, and the reaction from this at A is:

$$A_2=\frac{lM^a}{2}\,\frac{2}{3}+\frac{lM^b}{2}\,\frac{1}{3}=\frac{l(2M_a+M_b)}{6}.$$

The resultant reaction at a is then:

$$A=A_1-A_2=\frac{Ml}{3}-\frac{l(2M_a+M_b)}{6}.$$

The moment of inertia of the beam being I_B, we have:

$$\alpha_1=\frac{A}{EI_B}=\frac{Ml}{3EI_B}-\frac{l(2M_a+M_b)}{6EI_B} \quad . \quad . \quad . \quad (3)$$

In a similar way we get the angular deflection of the beam at B:

$$\beta_1=\frac{B}{EI_B}=\frac{Ml}{3EI_B}-\frac{l(M_a+2M_b)}{6EI_B} \quad . \quad . \quad . \quad (4)$$

As the beam is rigidly connected to the columns at A and B, it follows that the angular deflections at those points must be of the same size for the beam and columns, and we must, therefore, have: $\alpha=\alpha_1$ and $\beta=\beta_1$.

For the column at A we have:

$\alpha=\frac{hM_a}{4EI_c}$, and for the column at B: $\beta=\frac{h_1M_b}{4EI_c}$, and putting $\alpha=\alpha_1$ and $\beta=\beta_1$, we get:

$$\frac{hM_a}{4EI_c}=\frac{Ml}{3EI_B}-\frac{l(2M_a+M_b)}{6EI_B} \quad . \quad . \quad . \qquad (5)$$

$$\frac{h_1M_b}{4EI_c}=\frac{Ml}{3EI^B}-\frac{l(M_a+2M_b)}{6EI_B} \quad . \quad . \quad . \qquad (6)$$

In most cases the columns are of the same height and same size, and we then have, on account of the symmetry, $M_a=M_b$. Putting $M_a=M_b$ in (5), we get:

$$\frac{hM_a}{4EI_c}=\frac{Ml}{3EI_B}-\frac{M_al}{2EI_B} \quad \text{or} \quad M_a\left(\frac{h}{4I_c}+\frac{l}{2I_B}\right)=\frac{Ml}{3I_B},$$

and then:

$$M_a=\frac{Ml}{3I_B\left(\frac{h}{4I_c}+\frac{l}{2I_B}\right)}=\frac{M}{\frac{3hI_B}{4lI_c}+1.5}$$

and putting

$$\frac{3hI_B}{4lI_c}=C$$

$$M_a=\frac{M}{1{\cdot}5+C} \quad . \quad . \quad . \quad . \quad . \qquad (7)$$

We have now found the value of M_a expressed by M and C.

From the expression for C it is seen that C is a constant dependent on the dimensions of the columns and beam, and those dimensions must therefore be known before M_a can be found. The expression for C shows that the larger the beam is in proportion to the columns, and the shorter its length in proportion to the height of the columns, the smaller M_a will be.

If the columns are very large in proportion to the beam, C will be small, and M_a will reach its maximum value, for $C=O$. Of course, C can never reach the value of zero, but it may be so small as to be practically negligible.

Putting $C=O$, we get:

$$M_a=\frac{M}{1{\cdot}5}=\frac{Wl}{8\times1{\cdot}5}=\frac{Wl}{12},$$

which means that the beam in this case is fully fixed at the ends, and the maximum bending moment at the centre of the beam will then be:

$$M_1=M-M_a=\frac{Wl}{8}-\frac{Wl}{12}=\frac{Wl}{24}.$$

For use in practical designing the formula (7) may be employed in different ways according to the nature of the case in hand. If, in the case considered, the beam will evidently be large compared to the columns, the beam is first designed for a bending moment of $\frac{Wl}{8}$ at the centre, and the columns designed for direct compression, using a rather low working stress. I_B, I_c, h and l are then known, and C and M_a may be found.

The beam is then redesigned for a bending moment of $\frac{Wl}{8}-M_a$ at the centre, and the columns for the direct compression and the moment M_a acting at the top. In many cases it may be advantageous, and will also simplify the calculations, first to decide the value of M_a.

For instance, M_a may be taken as $\frac{Wi}{16}$, and the bending moment at the centre of the beam will then be $\frac{Wl}{8}-\frac{Wl}{16}=\frac{Wl}{16}$, or equal to M_a, and the beam is then designed according to those bending moments. From (7) we get:

$$1{\cdot}5+C=\frac{M}{M_a}=\frac{\frac{Wl}{3}}{\frac{Wl}{16}}=2 \quad \text{or} \quad C=2-1{\cdot}5=0{\cdot}5,$$

and as

$$C=\frac{3hI_B}{4lI_c}, \; I_c=\frac{3hI_B}{4lC}=\frac{3hI_B}{2l},$$

which gives the moment of inertia of the columns.

In regard to the columns, they have to be designed for a central load P, being the reaction from the beam and for the bending moment M_a, acting at the top of the beam. If the cross sectional area of the column is s, and the distance from the neutral axis to the extreme fibres is e, we have the stress in the top section of the column:

$$C=\frac{P}{s}\pm\frac{M_a e}{I_c}.$$

We have so far supposed that the columns supporting the beam are of same height, but should not this be the case M_a and M_b will be different, and we then have the equations (5) and (6) to solve the problem.

Those equations may be solved for M_a and M_b, but the method then leads to rather complicated calculations, both in regard to columns and beam.

An easier method to solve the problem in this case is to try and get M_a to be equal to M_b, as the beam then will be symmetrical as when the columns are of same height. That M_a is equal to M_b means that the angular deflections of the two ends of the beam will be of same size, but in that case must also the angular deflections of the columns at A and B be of equal size, or we must have

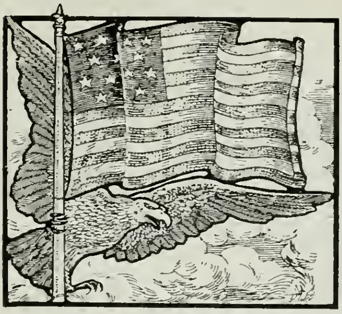
$$\alpha=\beta \quad \text{or} \quad \frac{hM_a}{4EI_c}=\frac{h_1M_b}{4EI_c},$$

and as $M_a=M_b$

$$\frac{h}{I_c}=\frac{h_1}{I_{c1}}.$$

From this it will be seen, that if the proportions between the heights of the columns and their moments of inertia are of equal size, the bending moments M_a and M_b acting on the ends of the beam will also be of equal size. For columns of different heights we can therefore make the calculations in the same way as for columns of equal heights, and use, for instance, the length of the shorter column in the calculations; only, it must then be remembered, that the moment of inertia of the longer columns must afterwards be increased in the same ratio as it is longer than the shorter column.

(To be continued).



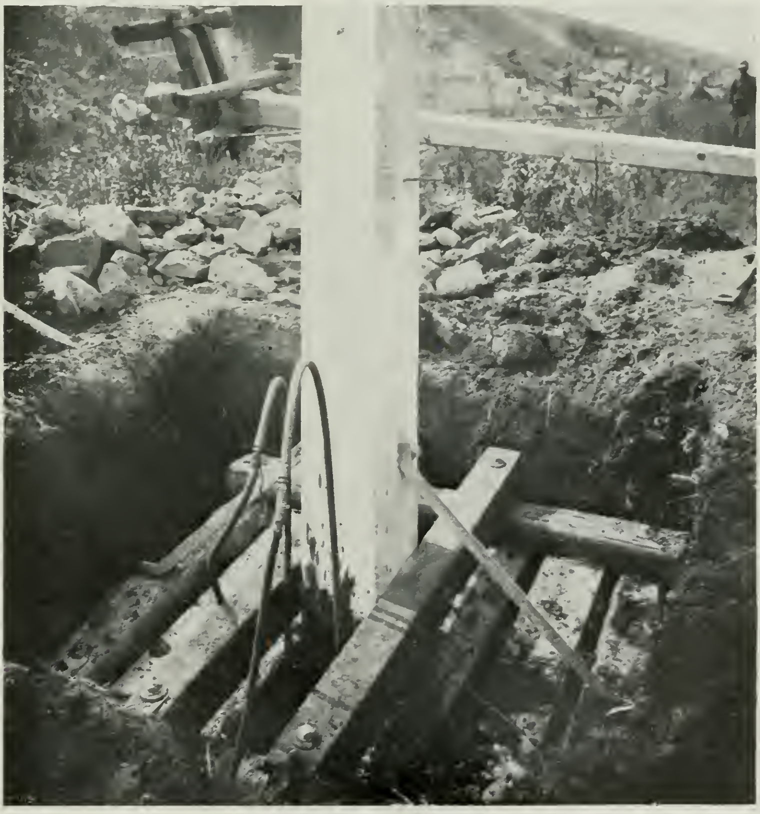
CONCRETE AS DISCUSSED AT THE NATIONAL ASSOCIATION OF CEMENT USERS.

The Eighth Annual Convention of the National Association of Cement Users of America was held at Kansas City in March last, when numerous exceedingly useful papers and reports were presented. We are giving two of these in the current number, and shall publish several others in future issues.—ED.

CONCRETE TELEGRAPH POLES ACROSS MARSHY GROUND

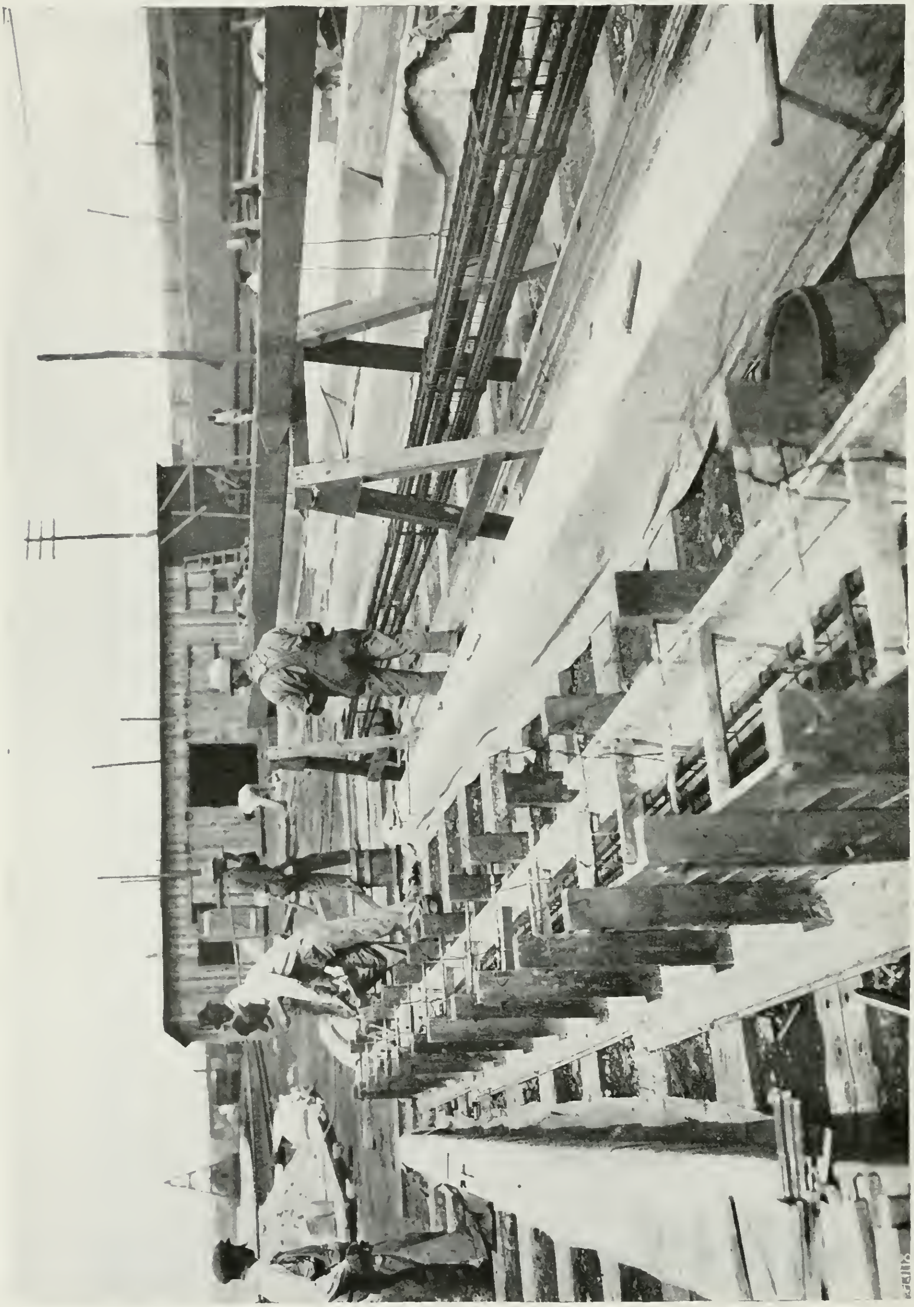
By GEORGE GIBBS, Consulting Engineer, Pennsylvania Railroad.

Among the facilities required on the New York Terminal Division of the Pennsylvania Railroad was the Railroad Company's telephone service. The line for this purpose



Setting a Telegraph Pole.
Concrete Telegraph Poles Across Marshy Ground

forms an extension of the New York-Philadelphia main line, from its point of intersection with the Terminal Division at the east side of the Passaic River near Newark,



Reinforced Concrete Telegraph Poles Across Marshy Ground.

Manufacture of Poles, Showing Frame in Form Ready for Pouring and Skeleton Frame Suspended in Place.

N.J., and consists of a pole line along the right of way across a continuous five mile stretch of semi-tidal meadow swamp land known as the "Hackensack Meadows," to the tunnel portal at Bergen Hill, the Hackensack River being crossed midway of the section. The ground surface is covered with a heavy growth of reeds, and the top stratum is a peaty bog, from 8 to 15 ft. deep, underlaid with varying strata of clay, fine sand, and mixed sand and clay for very considerable depths. It will be seen, therefore, that the problem was to provide a design of pole which would not only be strong in itself, but which could be set in a stable foundation in a soft and uncertain ground. It was desired to make the line entirely secure against interruption by severe storms or by fires in the swamp reeds. It appeared, under these conditions, that the ordinary type of wooden pole would not be wholly satisfactory, either as regards strength for such a heavy service or safety as regards the frequent meadow fires.

The use of reinforced concrete poles for the purpose appealed to us, but very little reliable data about either the cost or service from such design was obtainable. In Europe the author had observed the quite extensive use of concrete poles, chiefly for electric transmission purposes, but details of their construction were not at hand. These poles are frequently of structural steel covered with a protective coating of concrete, and sometimes are of wood encased in concrete, and much ingenuity has been expended in devising special forms of composite poles, but with what practical success it is difficult yet to say.

The desirability, however, of securing a strong fireproof line across this section of our road was so great that the writer undertook to design in his office a simple form of reinforced concrete pole, which could be manufactured on or near the work and at a cost which would not be prohibitive. Without previous data to go upon the design evolved is somewhat experimental, and probably the notes here given will not be of very great assistance in fixing a precedent for an ordinary telegraph pole, as our conditions called for heavier construction than usually required of such lines. The data, however, is given for what it is worth, as indicating a heavy and difficult piece of construction for a special problem.

There were 202 poles required, spaced from 70 to 135 ft. apart, with an average standard spacing of 120 ft., the variations in span being due to numerous railway and highway crossings. The height of the top of poles above the ground also varies for the local conditions from 25 to 50 ft., and the total length of poles from 35 to 65 ft. The loading called for an ultimate capacity of 60 open telegraph wires and two 40-pair telephone cables and one signal control cable. The open wires are of copper, Nos. 7 and 8 B. & S. gauge, and the cables Nos. 13 and 16 B. & S. gauge insulated wire, with a lead covering. The cables are carried by steel messenger wires. The weight of the wires and cables is 10½ pounds per running foot, giving a total loading for the average span of 1,268 pounds per pole, or including cross arms, insulators, brackets and fittings, 1,500 pounds per pole. In addition to this static vertical loading, emergency conditions were assumed of an extra loading of half inch of ice on the wires and a wind pressure of 8 pounds per sq. ft. on wires and 13 pounds per sq. ft. on the poles. The stresses under these conditions not to exceed the elastic limits of the wires and of the carrying structures. These assumptions are equivalent to a vertical loading of 7,600 pounds per pole and a horizontal force of 4,000 pounds exerted one foot below the pole top. The normal sag of the wires at 60 degrees Fahrenheit temperature is 4½ in. and a normal tension of 273 pounds for No. 8 wires and 130 pounds for No. 9. The maximum hot sag at 130 degrees Fahrenheit is 11 in.; the tension in the wires under maximum wind pressure with the ice loading at 20 degrees Fahrenheit is 702 pounds and 470 pounds respectively for the two sizes of wires.

The design adopted is that of a tapering, smooth surface, reinforced concrete pole, square in cross section with chamfered corners and having a taper of ½ in. in 5 ft. The concrete is of 1½ : 2 : 4 mixture, assumed to have an ultimate strength in compression of 2,000 pounds per square inch. The reinforcement is composed of mechanical bond bars tied together into a square skeleton frame, and in the complete pole this reinforcement is covered by a one-inch minimum thickness of concrete. The following table gives details of the complete construction.

In order to manufacture the poles economically a yard was established adjacent to the right of way and with side track connections for delivery of materials. The ground was levelled and set with stringer pieces, 3 ft. apart, of old wooden car sills, so as to furnish a convenient surface upon which to erect the timber forms. This yard permitted of the simultaneous manufacture of six poles and the storing in place of ninety poles during the period of seasoning. In the process of manufacture the skeleton reinforcement is first made up, the twisted steel mechanical bond bars being tied together to form a square skeleton frame; this frame is then suspended at the proper height above the ground and the sectional wooden frames put in position around it. The concrete mixture is then poured in and carefully tamped. In this way six poles were made per day and were left in place at least sixteen days to season.

TABLE OF CONCRETE POLES.

Length, feet.	Equivalent pull, 1 ft. from top.	Edge distance for steel.	Area steel.	Extreme Fibre stress in concrete compres'n.	Unit compression stress in steel.	Weight of pole.	Cubic ft. concrete.	Weight of steel, pounds.	Lin. ft. of wire wrapped around pole.
30	4066	1·75	3·38	1953	19059	5000	33·15	819	228
35	4321	1·75	3·63	2087	20480	6200	41·44	1077	273
40	4505	1·75	4·03	1917	19667	7600	50·75	1309	333
45	4647	1·75	4·40	1987	20428	9200	61·58	1493	375
50	4763	1·87	4·73	1886	19692	11000	73·24	1792	431
55	4860	1·87	5·00	1832	19155	13000	86·58	1974	490
60	4945	2·00	5·30	1812	19083	15000	101·28	2249	533
65	5020	2·00	5·48	1976	18591	17600	117·23	2575	618

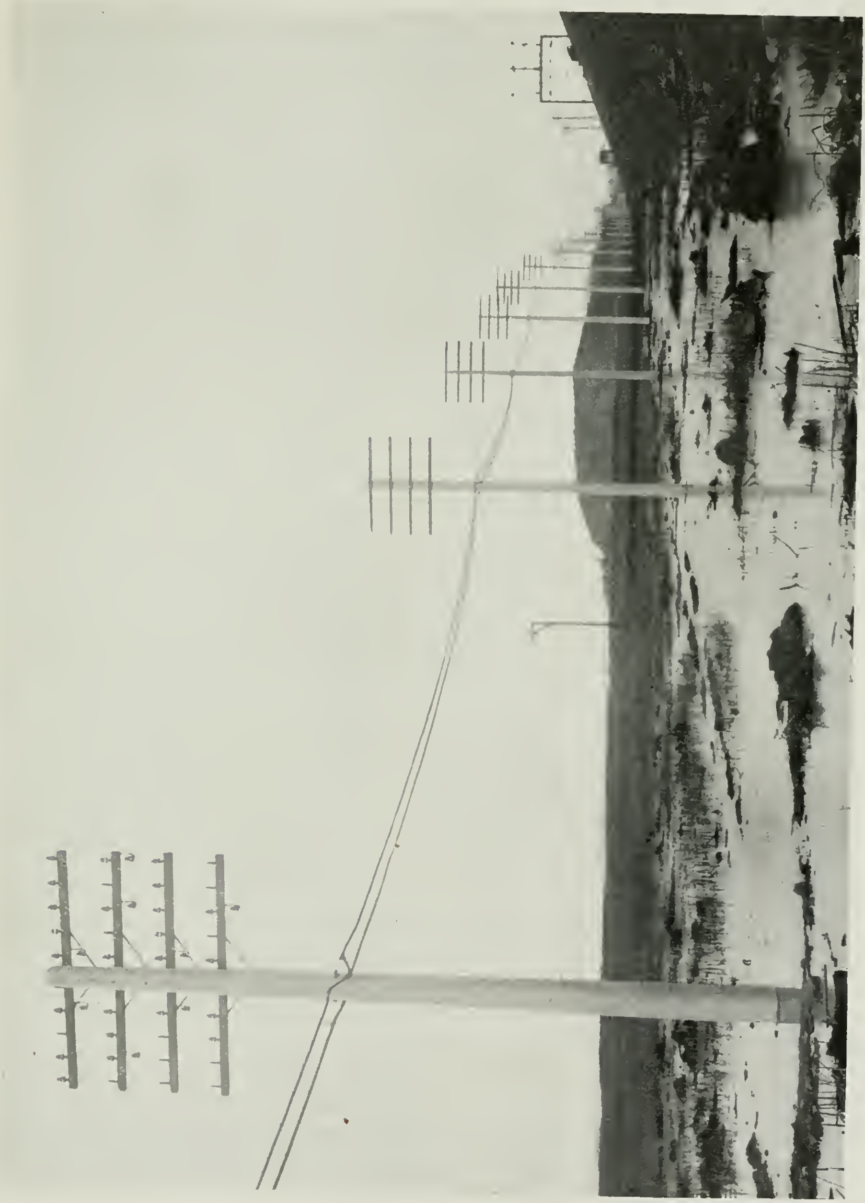
After a number of experiments it was found best to set the poles in pits excavated in the marshy stratum. These pits were generally about 9 ft. square and 5 ft. deep, and a timber grillage was placed around the base of each pole, and about 5 ft. below the top of the ground. This grillage consisted of six track cross-ties bolted together and to the pole, and partly planked over by 3-in. rough lumber. The pole, which projected below the grillage and was pointed at the butt, was jetted down by compressed air into the sandy layer, so that the grillage would rest at the bottom of the pit. The pits were then back-filled with rock and clay. Poles on curves are cross-guyed, and the terminal and railway crossing poles are head-guyed with steel cables.

In order to determine the stability of the foundations, as above described, as well as the transverse strength of the pole itself, a test was made of a 55-ft. pole set in a 13-ft. deep foundation. The test consisted in applying a horizontal pull 39·5 ft. from the ground. The breaking load was 4,360 pounds and the point of fracture 20 ft. from the ground, at the top of the 32-ft. reinforcement bars. The foundation itself was found to be amply secure.

The actual cost of labour and material for manufacture of these poles, reduced to a unit cost for an average length pole of 40 ft., weighing 7,600 pounds, was as follows :—

Forms—labour and material	$19.00
Concrete—labour and material	26.50
Reinforcement—labour and material	26.50
Total per pole	$77.50

The above does not include cost of preparing site, engineering, etc., and is for the bare poles (without fittings or foundation timbers). To obtain the cost of the poles erected, there must be added the cost of loading, distribution, foundations and setting, cross arms and fittings, but these items were special to the particular location and would be of little use as a matter of general information. It might be stated, however, that the cost of handling and erecting these heavy poles was considerably

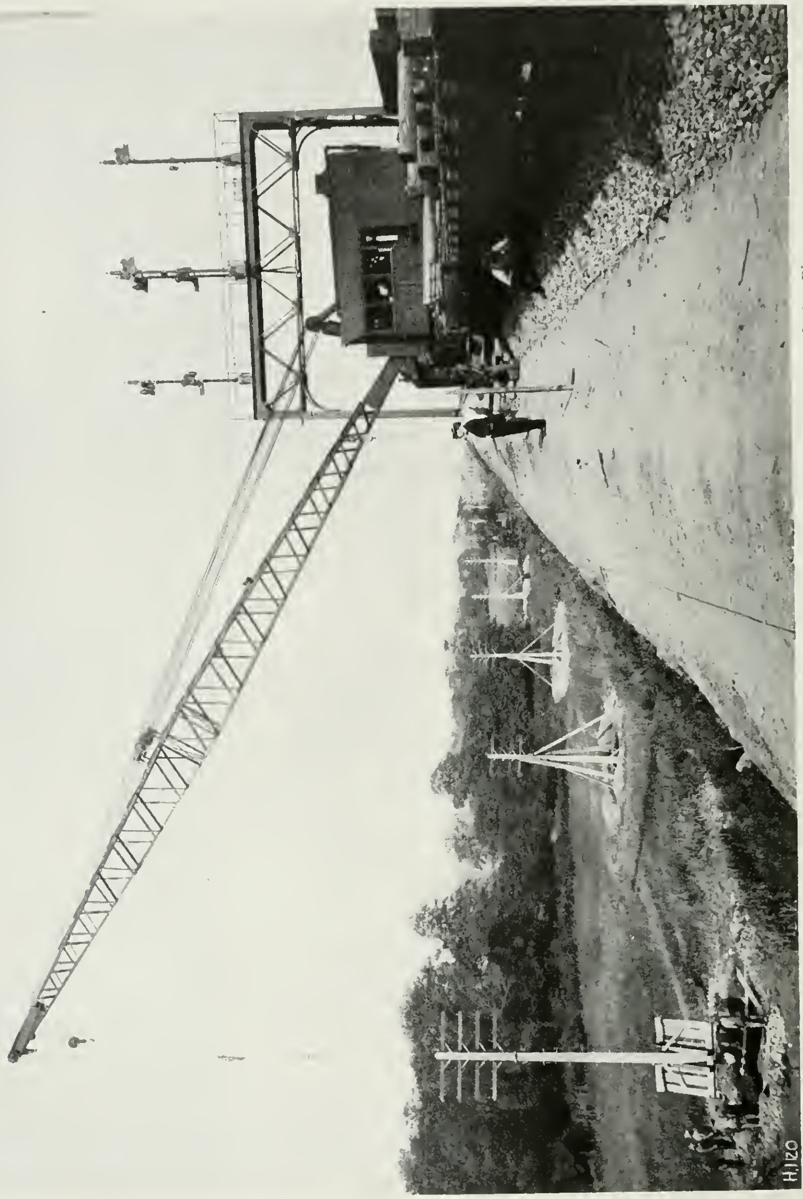


Concrete Telegraph Poles Across Marshy Ground

A View of Some of the Poles Across Hackensack Meadows, N.J.

more than would be the case for wooden poles; furthermore, care must be taken in loading and unloading them and special appliances are required for their erection.

In the special case in question it was figured that the cost of the line erected complete was not greatly in excess of that of an equally stable wooden pole line, as in order to meet such a condition it would have been necessary to employ "H" frame poles and set them in equally expensive foundation pits. In the case of this line it is



Erection of Pole Line, Showing 90 ft. Boom Derrick Lifting Pole Over Foundation Pit
CONCRETE TELEGRAPH POLES ACROSS MARSHY GROUND

interesting to note that the foundations and setting cost considerably more than the poles themselves.

To sum up, it may be said that for the particular case of this heavy line having very difficult foundation conditions, the use of a reinforced concrete pole resulted in a durable, safe and neat appearing construction at little additional cost to that of a wooden pole line inferior in all these respects.

REINFORCED CONCRETE PILES.

By ROBERT A. CUMMINGS, M.Am.Soc.C.E.

It is well established that concrete piles are a satisfactory substitute for wooden piles in increasing the stability of foundations, whether they are of the "cast-and-driven" type or the "cast-in-place" type. The "cast-and-driven" type are made of reinforced concrete, moulded to the desired shape and cured before being driven. The "cast-in-place" type are made by forming a hole in the ground and filling it with concrete.

The history of the "cast-and-driven" type is intimately connected with the development of reinforced concrete, and in the same manner has been embarrassed by patent litigation. In March, 1907, a final decision was reached in the British High Courts of Justice that the fundamental idea of the reinforced concrete pile was covered by Brannon's patent of 1871, and that subsequent improvements must be limited to the details of reinforcing.

At the Chicago Convention of the National Association of Cement Users in 1909 the writer described certain tests and methods of reinforcing for increasing the unit value of concrete in compression. It is the application of one of these methods and other practical improvements that form the subject of this paper.

Method of Reinforcement.—All piles of the "cast-and-driven" type must be reinforced with longitudinal rods, because the pile is hoisted and handled by a line from the pile-driver, which is fastened at or near the butt. Consequently, the pile has to sustain its own dead weight by being raised, as well as shocks and impact against obstacles, before reaching its position in the leads of the pile-driving machine. With the point on the ground and the other end being elevated, the pile must act and be designed as a beam supporting its own dead weight and shocks. The limiting proportions of depth or thickness of pile to the unsupported beam length produces heavy tension and compressive stresses, with considerable deflection.

It has been observed that when handling a pile of this type, it rarely fails in compression in the concrete, but cracks are usually discovered on the tension side. These cracks can be accounted for by the slipping of the usual longitudinal rods used in the concrete while the pile is being hoisted. While such cracks are not sufficiently serious to condemn the pile, they may affect the permanency of the reinforcement.

In order to overcome this defect all longitudinal rods should be anchored at the ends, those at the butt opposite each other being bent over into a loop and welded together, while those at the point are all brought together and electrically welded into one piece. Twisted and deformed rods are advantageous for longitudinal reinforcing as the allowable bond stress is higher than for plain rods.

The uniform circumferential spacing of longitudinal rods is very important, because any side of the pile may be subjected to tensional stresses, and whichever side is in tension there must be sufficient steel in position to take the strain. The circumferential spacing of the longitudinal rods can be secured by means of a special spacing device placed at intervals of about 5 ft. throughout the length of the pile.

The hooping of concrete adds greatly to its ability to resist axial loads. Therefore, longitudinal reinforcing should have a helical wrapping of wire throughout the length of the pile, the pitch of which must not exceed 3 in. This wire wrapping will assist in taking care of diagonal stresses resulting from the handling of the pile.

Practical experience indicates that the butt end of the pile which receives the impact of the hammer should be especially reinforced. This has been done by means of a special reinforcement consisting of a unit cage of flat bands, held 2 in. on centres by a spacing bar for a distance of 2 ft. In the plane of each band a flat wire spiral is fastened to the cage. The embedment of this unit cage in the butt of the pile forms a resilient cushion to receive the impact of the hammer. In no case, in the driving of thousands of piles with this cushion, has the butt itself been broken.

Methods of Driving.— Piles of the "cast-and-driven" type are handled and driven by means of an ordinary pile-driver. The ability to vary the falls of the drop-hammer is of great utility in overcoming the variable resistance to be met with in the driving. It has been found advisable to increase the weight of the ordinary drop-hammer in ratio of weight of hammer to weight of pile of from 2 or 3 to 1, so that the weight of the drop-hammer for driving the concrete piles will vary from 7,000 to 12,000 pounds.

Steam-hammers are not as efficient or desirable for driving concrete piles as are drop-hammers. This was shown last autumn on a contract when a test was made between a steam-hammer and a heavy drop-hammer, under the same conditions, using the same kind and size of concrete pile. The heavy drop-hammer did not break a pile, whereas the steam-hammer broke several below the cushion. Further, the steam-hammer did not drive as many piles in a given time as did the drop-hammer.

The following explanation is offered as it applies also to the driving of the heavy steel casings or core used in the making of the "cast-in-place" type of pile. The limited fall (3 ft.) and light ram (3,000 pounds) of the steam-hammer, while delivering twice as many blows per minute as the drop-hammer, loses a large part of its energy in overcoming the proportionately heavier weight of the pile, steel casing or core.

The following analytical treatment by Mr. Barton H. Coffey, M.E., of New York, confirms and justifies the practice of the writer in using hammers as heavy as 12,000 pounds for driving concrete piles, and it may be asserted with some feeling of confidence that, if penetration is to be the gauge for measuring the supporting power of a pile, the ratio of the weight of the hammer to the weight of the pile or core that is driven must be taken into consideration.

The advantage of using a heavy hammer is evident from the following analysis :—

Let M equal Mass of Hammer $= \frac{w}{g}$
,, W ,, Wt. ,, ,,
,, X ,, Wt. or mass of pile in per cent. of hammer.
,, V ,, Velocity of hammer on striking pile.
,, V_1 ,, Common velocity of hammer and pile.
Momentum of hammer $= MV$ (on striking pile).
,, ,, ,, and pile at common velocity $= M\ (1 \times X)\ V_1$.

These are equal provided no external force acts, which we will assume for the present is the case.

$$\text{Then } MV = M(1+X)V_1 \quad V_1 = \frac{V}{1+X} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{I}).$$

The energy in hammer on striking pile is—

$$1/2\ MV^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{II}).$$

The energy in both hammer and pile at common velocity is—

$1/2\ M\ (1X)\ V_1^2$, which upon substituting (I) becomes

$$1\ 2\ M\frac{V^2}{1+X} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{III}).$$

The difference between (II) and (III) represents the *loss of kinetic* energy in the system at point of common velocity or greatest compression. In other words, the percentage of the original energy of hammer that has gone into compressing pile and hammer, distorting or crushing them, and into heat broadly-internal work-done upon each other by the striking bodies. Obviously, the smaller this percentage of internal work the less liability there is of crushing or distorting the pile. To resume—

$$(II)-(III)=1/2MV^2-1/2M\frac{V''^2}{1+X}=1/2MV^2\left(1-\frac{1}{1+X}\right)1V$$

equals the internal work.

The following table gives internal work for various values of X in per cent. of the total kinetic energy of the hammer :—

X	Internal Work.
0·25	20 per cent.
0·50	33 ,, ,,
0·75	43 ,, ,,
1·00	50 ,, ,,
1·50	60 ,, ,,
2·00	67 ,, ,,
3·00	75 ,, ,,

If your hammer weighs four tons and the pile one ton, there will be 20 per cent. internal work at maximum compression, whereas if the pile weighs eight tons the internal work will be 67 per cent.

There is an external force acting against the pile, *i.e.*, the frictional and displacement resistances of the earth.

Two extreme cases may be assumed limiting all others :—1st—There is no external force. There the internal work will be simply that necessary to overcome the inertia of the pile and put it in motion. In this case the table is rigorously accurate. 2nd—The external force is great enough to prevent any movement of the pile. In this case the entire kinetic energy of the hammer goes into the internal work, and the relative weights of hammer and pile are immaterial. All intermediate cases where movement occurs are a combination of 1 and 2, where obviously it is advantageous to employ a heavy hammer.

Manufacture.— The procedure in constructing "cast-and-driven" piles commences with the preparation of the moulding bed. This, of course, will vary with the site, but it is desirable to select a flat and convenient location near the place of driving. It is very important that the bed shall be stable, so that settlement due to the weight of the piles is avoided. Where the ground is soft and yielding, pine stakes 2 in. by 4 in. by 3 ft. long, pointed at the ends, are driven to a solid bearing. These stakes are located at intervals of about 4 ft. in each direction, and the tops cut to a uniform level. Then, 4-in. by 4-in. pine sills are toe-nailed to the top of the stakes in longitudinal rows about 4 ft. on centres. Upon these sills a 2-in. solid wooden floor is placed, which forms the moulding bed. It is desirable that this bed shall be uniformly level to receive the forms for the piles.

The forms are made of two pieces of 2-in. by 8-in. dressed pine battened together and placed on edge to form the sides of the pile. The bevels or angles for tapered or octagonal piles are made by placing loose pieces of bevelled wooden strips at each corner of the form. The steel reinforcement is delivered on the work in factory-made units, so that it can be placed in the forms at once. When a reinforcing unit is suspended and centred in position, the concrete of a wet consistency is deposited and carefully puddled. As soon as the concrete of the pile has solidified the forms are stripped and used for making other piles. The number of forms required will vary with the quantity of piles to be made and the prospective salvage in the lumber.

The curing of concrete in the normal manner delays the driving of the piles for a period of not less than three weeks, although a greater length of time is desirable, especially in cool and damp weather. Therefore, unless a stock of cured piles is always on hand, it frequently occurs that this type of pile cannot be used at all, and resort is had to the use of the " cast-in-place " type. This practice is open to question on account of the inability of plain concrete to resist even moderate tension. In fact, it is almost axiomatic that all concrete piles must be reinforced. Every concrete pile is subjected to strains that induce very serious tensile stresses in the pile. Such stresses may result from superimposed loads or a lateral strain from the soil. Further, the making and storing of concrete piles for use at any time necessitates a large financial investment.

In order to avoid the above-mentioned objections, the writer has adopted the method of steam curing of " cast-and-driven " piles. This enables such piles to be made and driven within three or four days and places the speed of driving on the same basis as that of the " cast-in-place " type. The means used for steam curing will vary with circumstances, location of the work, speed required for delivery of the piles, the number needed, etc. During the past winter the writer has used concrete piles made and driven within ten days, and, as a result of this experience, confidently recommends that such piles can be made, cured, and driven immediately.

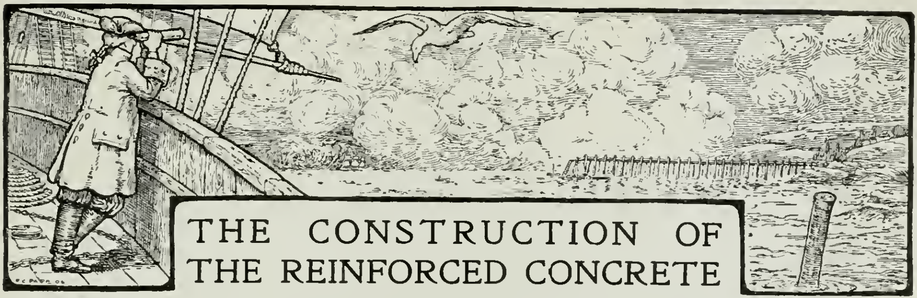
On the work with which the writer was connected the piles were allowed to set in a normal manner for five or six days, and were then gently hoisted from the moulding bed by a derrick, using an equalising spreader and bridle, the chains of the bridle being fastened so that the pile was balanced. They were then placed in stacks of twenty-five or thirty and separated from one another by wooden blocks, particular attention being given to securing a solid bearing for each pile.

A light wooden shed, practically steam-tight, was built entirely around the stack of piles. The steam was conducted direct from a boiler through a 1-in. pipe to three branch openings inside the shed. The steam pipe valve was opened and the piles were exposed to live steam for two or three days, when they were found to be ready for driving. On being first exposed to steam the moisture condensed on the surface of the piles and remained until absorbed by the concrete when the temperature of the steam was reached. This steam treatment should be distinguished from heat applied indirectly or baking, in accelerating the set and hardening the concrete. The writer sees no reason why boiling water should not be used for the same purpose if the conditions are favourable for adopting this method.

Precautions should be exercised in making sure that the concrete has solidified, and that it has received its initial set, before exposure to steam treatment.

The prospective field that is opened up by the steam treatment for the rapid curing of concrete seems to the writer to solve the difficulty incidental to the present methods of procedure in the construction of all classes of concrete structures.

The attention of the members of the Association is directed to the publication of the tests of the Structural Materials Laboratory of the United States Geological Survey, wherein it is conclusively shown " that a compressive strength considerably (in some cases over 100 per cent.) in excess of that obtained normally after ageing for six months may be obtained in two days by using steam pressure for curing mortar."

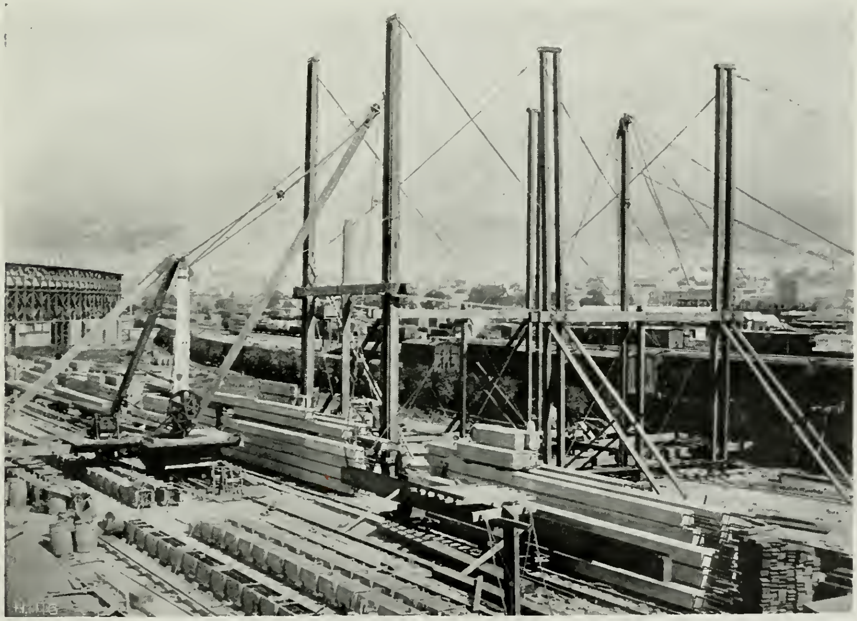


THE CONSTRUCTION OF THE REINFORCED CONCRETE PROMENADE PIER AT CAPE TOWN, SOUTH AFRICA.

In an article published in October last entitled "Some Examples of Reinforced Concrete Work in South Africa" we gave preliminary details of the construction of the pier at Cape Town, and we now present further particulars of this work. One of the principal features in connection with the construction of this pier was the use of the Piercy travelling stage in the driving of the piles, which is described below.—ED.

Much interest is being evinced throughout South Africa in the splendid growth of Cape Town's new pier, and the extremely rapid completion of the heavy pile-driving work for it has excited general comment.

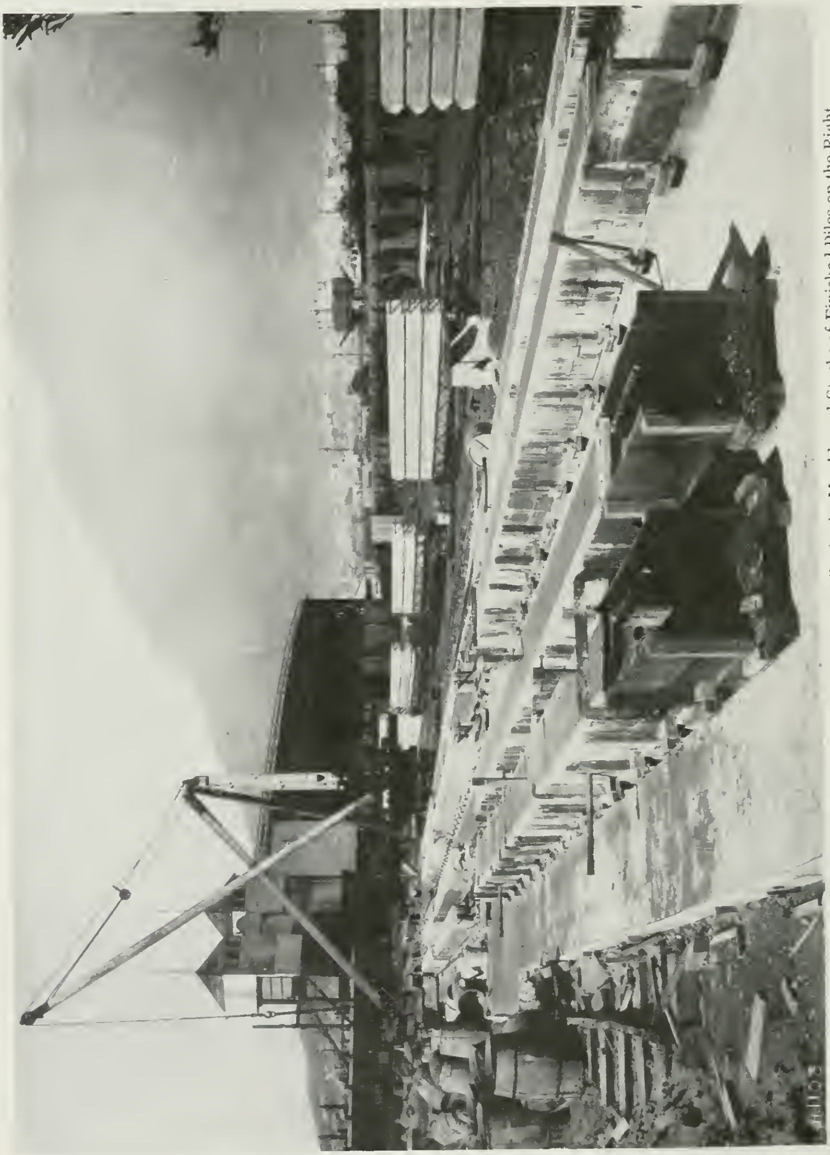
In our issue of October last we published a sketch plan of part of the pier and gave in outline a forecast of the general arrangement. We are again reproducing this plan in the present article on page 595. The pier is situated in a central position at the foot of Adderley Street, Cape Town, and reinforced concrete construction has been adopted throughout, including the retaining walls, bandstand, and other erections on the pier, and the pier itself is supported on reinforced concrete piles.



View showing Travelling Stage in Course of Erection, also Stacks of Reinforced Concrete Piles in Foreground.
Reinforced Concrete Promenade Pier at Cape Town, South Africa.

It will be seen that the total length of the new work is to be 1,500 ft., and that the first 300 ft. of this, 100 ft. wide, will be laid out with a roadway and footpath, and will be open to carriages and motors.

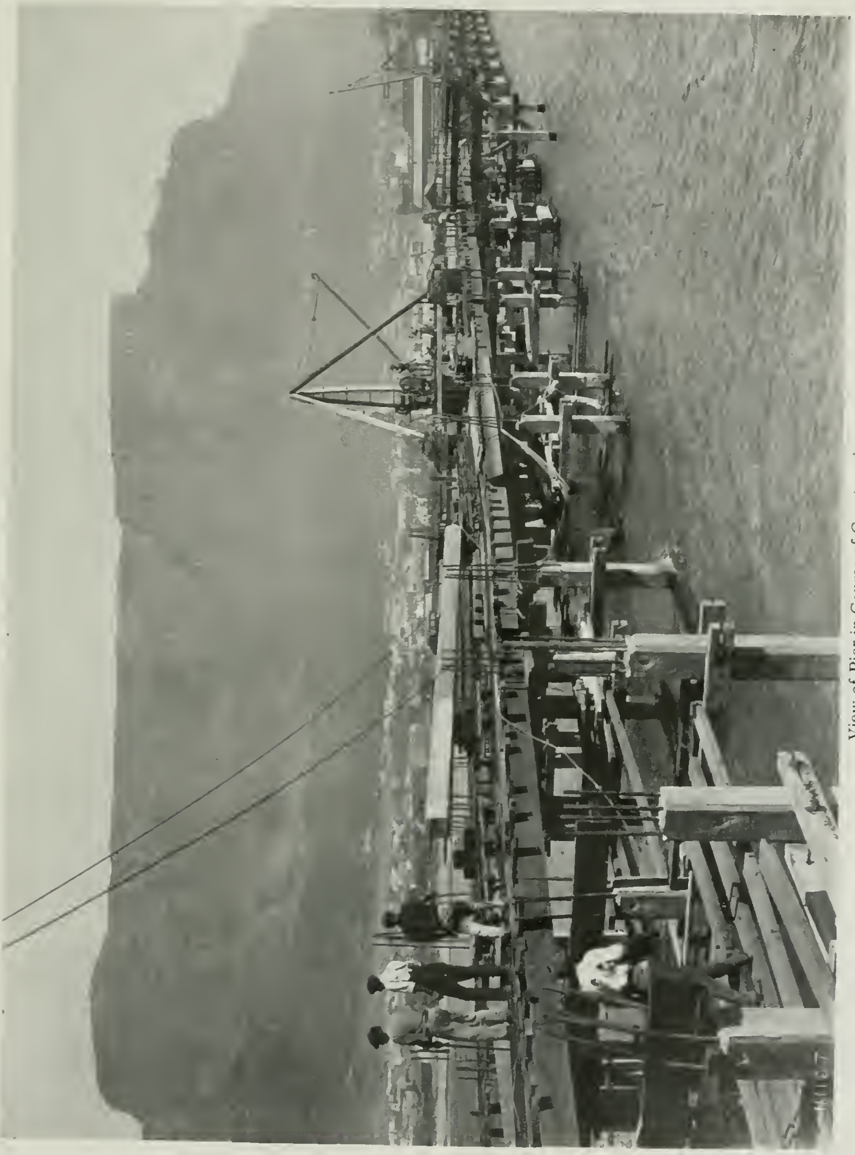
This approach leads to a circular space, partly built on piles and with reinforced concrete parapets, and also provided with footpaths and roadway, in the centre of which the statue of Van Riebeck will find a position worthy of the great pioneer, and facing Table Mountain. To the right the new marine



View of Pier Works Yard, Showing Reinforced Concrete Piles in the Moulds and Stacks of Finished Piles on the Right
Reinforced Concrete Promenade Pier at Cape Town, South Africa

drive, flanked by a fine belt of trees, will commence and encircle part of the bay. Beyond the circus the pier is continued for a length of 1,000 ft. and a minimum width of 45 ft., and is constructed entirely of reinforced concrete.

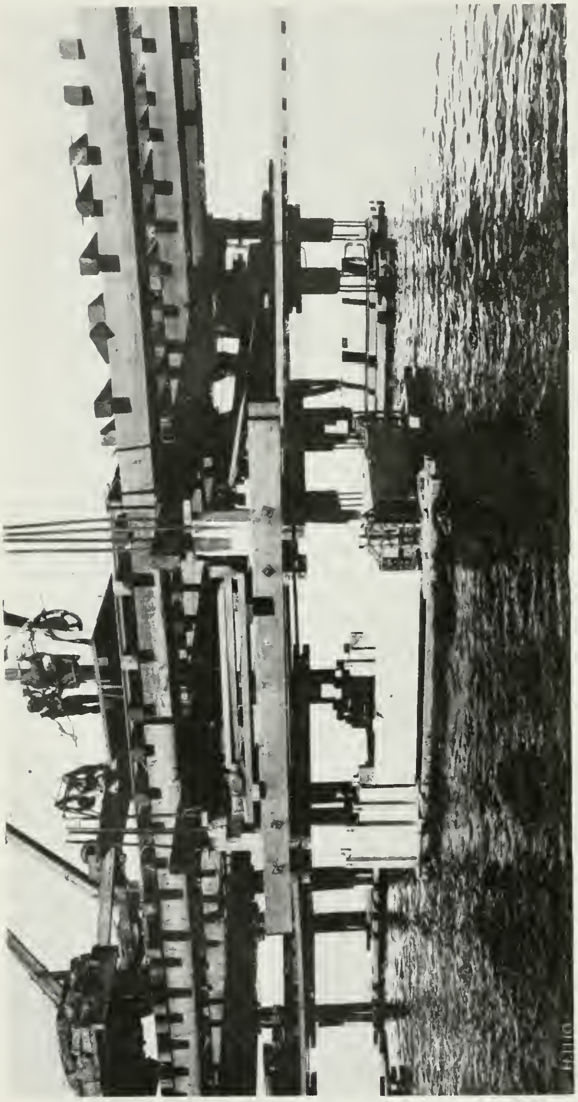
The whole of the work of driving the reinforced concrete piles for this part of the work was completed in March, having taken the extremely short time of four months, or about 900 working hours, with the aid only of the unskilled local labour available. No overtime or night shifts were resorted to, and many interruptions occurred through holidays and unfavourable weather.



View of Pier in Course of Construction.

Reinforced Concrete Promenade Pier at Cape Town, South Africa

This noteworthy result has been accomplished by mounting the pile-driver and all its machinery on a travelling stage specially built for the work, on Piercy's well-known principle, a description of which was given in our October issue. In this way all temporary staging and piling were entirely dispensed with, the self-contained travelling stage moving ahead of the work, quite independently of it, and driving the permanent reinforced concrete piles as it went. These were immediately braced together by reinforced concrete members in the usual way, a gangway for running the piles out of the pile-driver being formed by longitudinals resting on cross timbers laid on the tops of the piles.

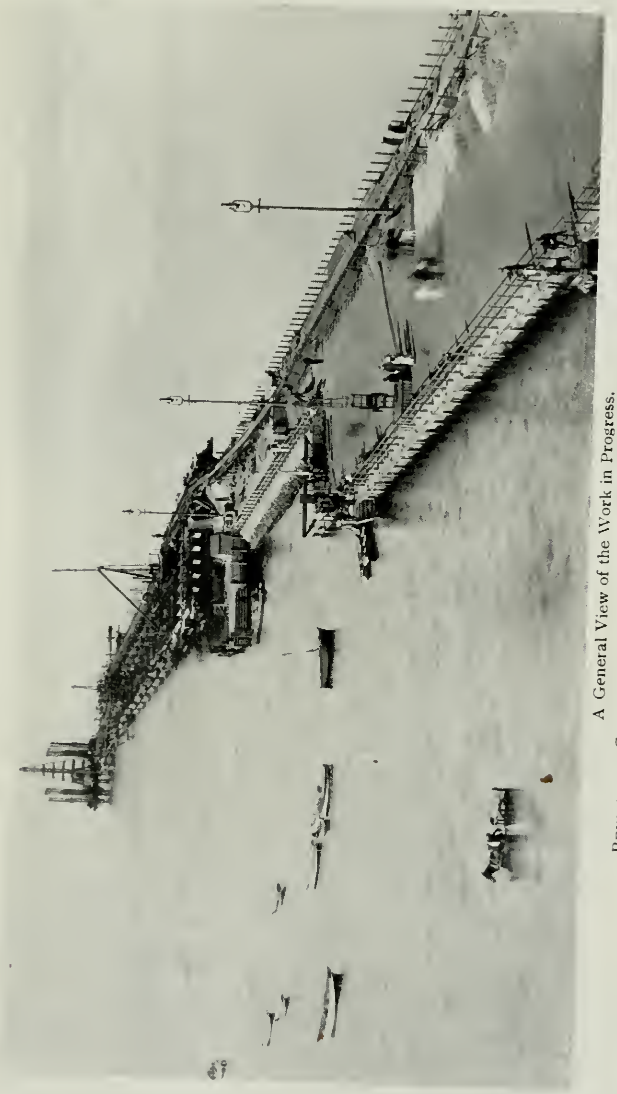


Side View of a Portion of the Pier, Showing Low Water Bracing Connections.
REINFORCED CONCRETE PROMENADE PIER AT CAPE TOWN, SOUTH AFRICA.

The stage was built in the pier works yard, of timber and steel, taking eleven days to erect, and then travelled, or "walked," on its own legs to its position in the sea for driving the first pile.

On its initial journey it traversed a distance of about 700 feet overland at an average speed of about 56 ft. per hour and of 500 ft. in the sea, the rate of travel here being 30 ft. per hour, owing to the spuds sinking further into the mud than on dry land. The stage negotiated a steep slope, with a difference in level of about 20 ft. before entering the water.

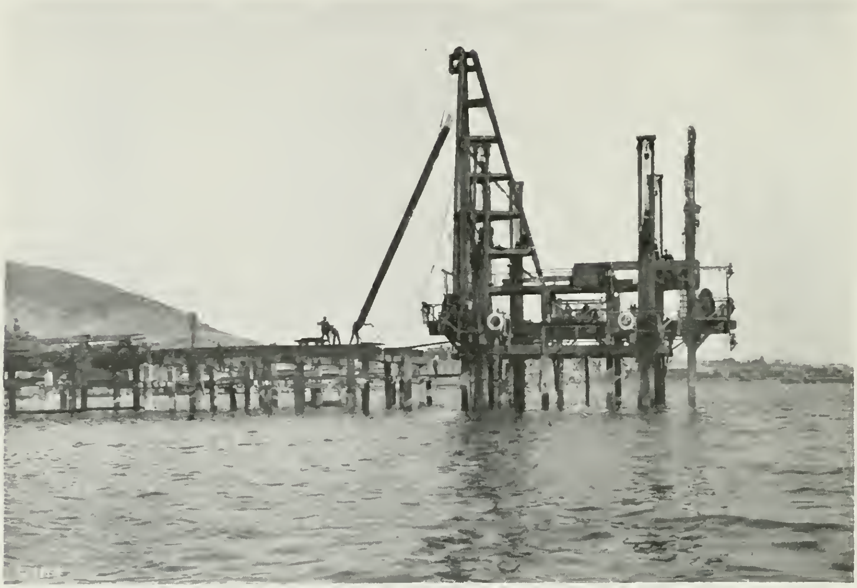
The piles are arranged in rows of four and were driven singly, the stage moving sideways or ahead after driving each pile, to bring the pile-driver into position for the next one. Its course was therefore a zig-zag one of right angle lines, reaching across the width of the pier. The ease with which these movements were made and with which the piles, weighing 5 tons, were swung across the gap between the travelling stage and the last row of piles driven excited universal admiration.



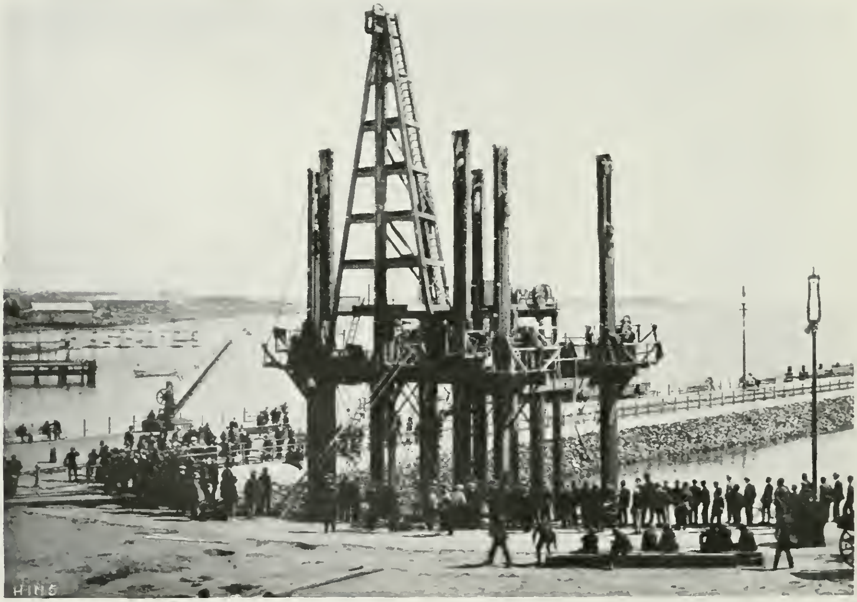
A General View of the Work in Progress.
Reinforced Concrete Promenade Pier at Cape Town, South Africa.

As mentioned in our issue of October, 1911, this stage can be erected in all security on shore, as shown on page 594, and walked out in the water to the work. The stage is moved ahead of the work as it proceeds. In the present case the stage consists essentially of two pairs of main girders, which are crossed and connected at the four points of intersection by four two-way roller carriages, thus enabling either pair of girders to support the other pair entirely while the stage is being moved. Each pair of girders is connected up and braced to form a square framework, at the corners of which the supporting spuds are situated. These spuds are raised or lowered as required, and together carry the whole stage, which is moved ahead, sideways, or slewed, as required by the pile driving.

The photographs show the stage and pile-driver and various views of its work. The adoption of such an appliance undoubtedly is evidence of a strong spirit of enterprise in the Corporation of the City of Cape Town, which is doing the work departmentally with its own staff, for although these travelling stages

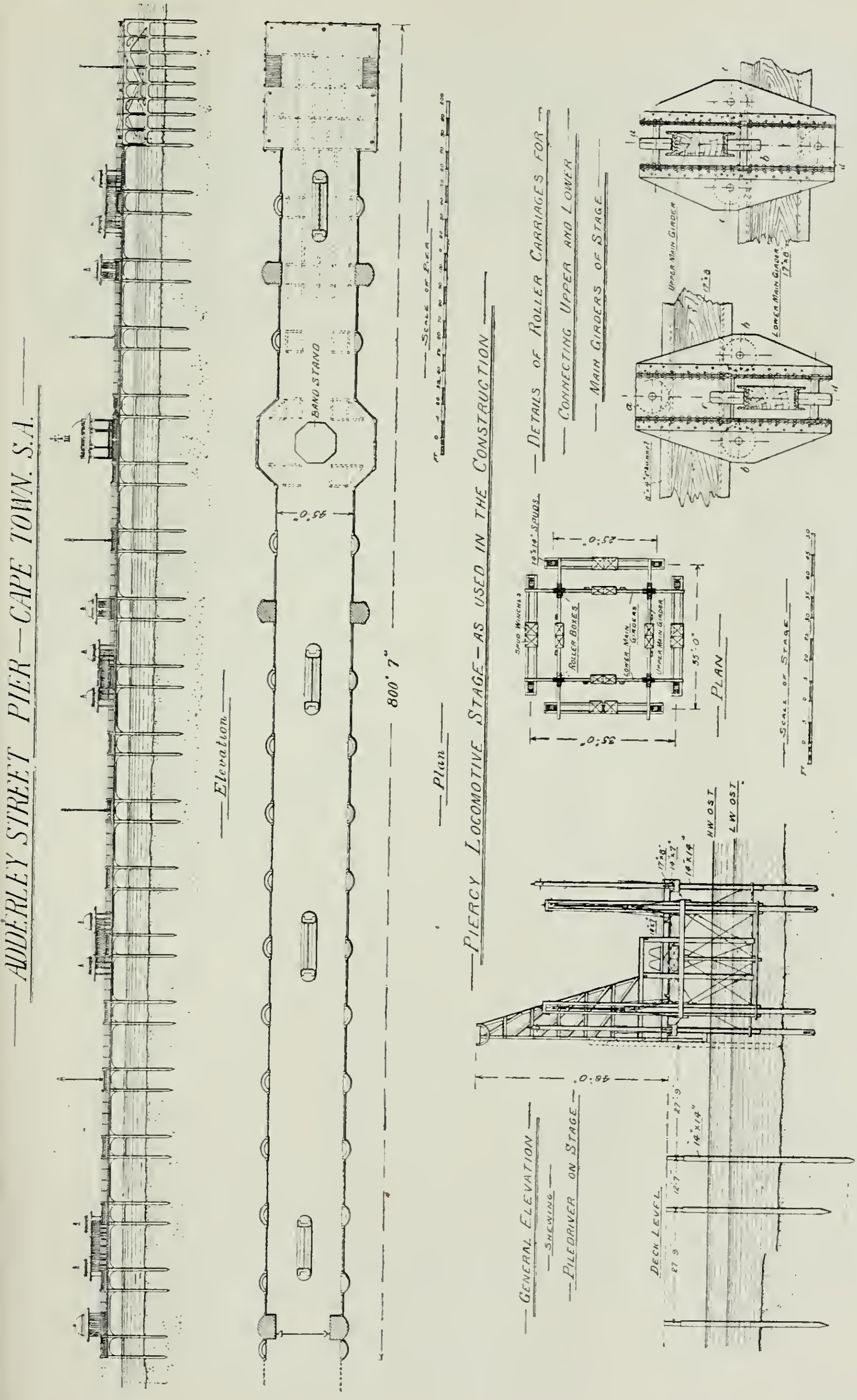


Reinforced Concrete Pile being lifted by the Travelling Stage preparatory to driving it.



View of Piercy Travelling Stage.

Reinforced Concrete Promenade Pier at Cape Town, South Africa.



Elevation, Plan, and Details of Pier.

Reinforced Concrete Promenade Pier at Cape Town, South Africa.

are now frequently used for large harbour works and submarine rock blasting, this is the first instance of its adaptation to pile-driving, and the Corporation deserves all credit for its courage in leading the way with such an important innovation.

There can be no doubt that the combination of long spans between the supports of a pier and a travelling stage such as was here employed for the pile-driving gives an ideal way of fully developing the inherent advantages of reinforced concrete for pier construction. One of the chief of these is the reduction in the number of supports—a most desirable object in sea work—to which reinforced concrete construction, with its almost unlimited facilities for making enormously strong piles, lends itself most admirably, but which advantage cannot easily be carried to its full limit when the cost of ordinary temporary staging has to be taken into account. Just where it is most advisable to use the least possible number of very strong supports, that is, in exposed positions, the cost of temporary staging would be correspondingly enhanced, and further increased by the necessity of making it strong enough to carry the heavy gear required for hoisting and driving very large reinforced concrete piles. All these difficulties disappear when the pile-driving machinery is mounted, as was done at Cape Town, on an independent travelling stage, and engineers will in future, no doubt, profit by the example set and the experience gained on this interesting piece of work.

RAILWAY PLATFORM WALLS.

By A CORRESPONDENT.

The following article on the cost of constructing railway platform walls which has been sent us by one of our subscribers, gives some interesting data on the various methods of constructing these walls, which should be of interest.—ED.

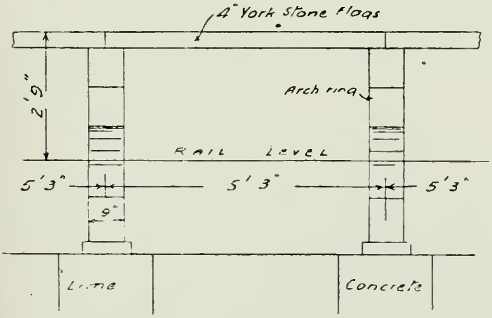
In estimating the cost of laying down a new line of railway, complete with stations, etc., it will be found that platforms add considerably to the cost.

Taking the case of an ordinary suburban station, on flat ground, with platforms, between ramps, 400 ft. long by 12 ft. wide, similar to *Fig.* 2, the cost is approximately £880.

Forty years ago the platforms at many of our terminal stations were built similar in type to *Fig.* 1. They had a 14-in. face wall, beyond which the platform flagging extended about 9 in. In many cases, instead of having the arches as shown, 9-in. cross-walls were built, forming a series of cells, over which the flags were laid.

— SECTIONAL ELEVATION. —

— FIG. 1. —



Estimated Cost 22s 4d per sq yard

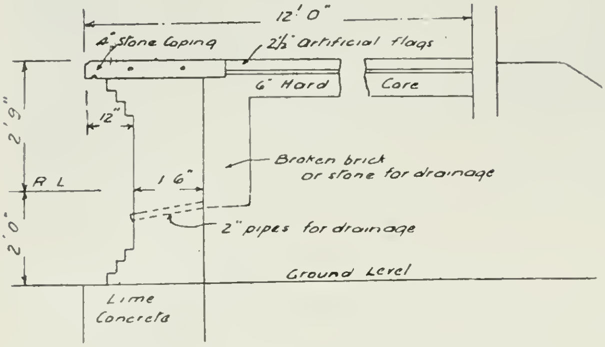
This type of platform cost approximately 22s. 4d. per sq. yd

Fig. 2 shows the more ordinary type of platform used by engineers, consisting of an 18-in. wall with a 4-in. coping projecting 12 in. from the face of wall and with two slate dowels to each joint. The coping has gradually increased in width from 12 in. until now 3 ft., mainly because it is a convenient width to trolley and trundle milk cans along.

The cost of this is 15s. 5d. per sq. yd., made up as follows:—

— SECTION. —

— FIG. 2. —



Cost 15s 5d per sq yard.
Wall only 33s 5d per lin· yard.
Without wall 8s 2d. per sq: yard.

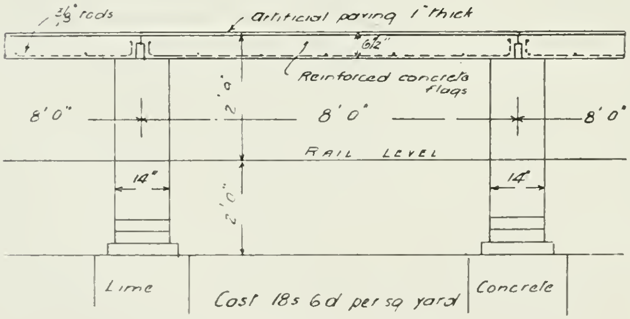
	£	s.	d.
·5 c. yd. Excavation @ 2s.	0	1	0
·42 c. yd. Lime concrete @ 12s. ...	0	5	0½
·74 c. yd. Brickwork @ 20s.	0	14	9½
1 lin. ft. Weep pipe @ 4d.	0	0	4
7 c. yd. Filling @ 8d.	0	4	8
·3 c. yd. Dry lining @ 3s. 6d. ...	0	1	1
1 s. yd. Coping @ 12s. 3d. ...	0	12	3
3 s. yd. Hard core @ 1s.	0	3	0
3 s. yd. 2½" Flags @ 6s. 6d.	0	19	6
	£3	1	8

Area, 12 ft. 0 in. × 3 ft. 0 in. = 4 s. yd. Cost, 15s. 5d. per s. yd.; wall only, 33s. 5d. per lin. yd.; platform, without wall, 8s. 2d. per s. yd.

In *Fig.* 3 is shown a platform very similar in design to *Fig.* 1. The walls are spaced 8 ft. centres and 14 in. in thickness, supporting reinforced concrete flags.

— SECTIONAL ELEVATION —

Live Load 2c per sq ft — FIG. 3. —



Cost 18s 6d per sq yard
Cost of 3ft width 18s 6d per lin yd

The flags can be either grooved or chequered, but this has a great disadvantage, unless water can be effectually drained off. The grooves, etc., hold the wet, which, when frozen, make the flags more liable to be slipped on.

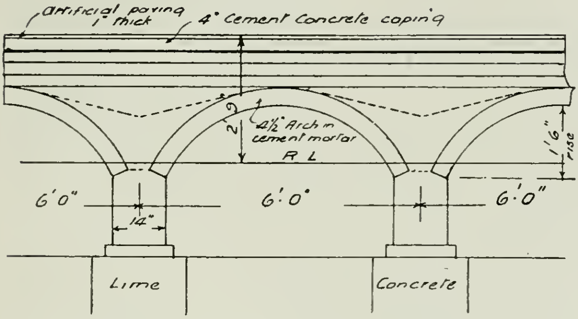
The following estimate of cost allows for an inch of patent pavement cement, which is taken to within 6 in. of the chamfered nose of the platform.

	£	s.	d.
1·4 c. yd. Excavation @ 2s.	0	2	9½
1·4 c. yd. Lime concrete @ 12s.	0	16	9½
2·2 c. yd. Brickwork @ 24s.	2	12	9½
1·7 c. yd. Cement concrete in flags @ 30s.	2	11	0
1·48 cwt. Steel rods @ 15s.	1	2	2
10·3 s. yd. Artificial pavement cement @ 5s.	2	11	6
	£9	17	0½

Area, 12 ft. 0 in. × 8 ft. 0 in. = 10·66 s. yd. Estimated cost, 18s. 6d. per s. yd.; 18s. 6d. per lin. yd. of 3 ft. width.

— ELEVATION. —

— FIG. 4. —


Artificial paving 1" thick
4" Cement Concrete coping
4½" Arch in cement mortar
2'
R L
1'6" rise
6'0"
14"
6'0"
6'0"
Lime
Concrete

Estimated cost 17s. 0d per sq: yard

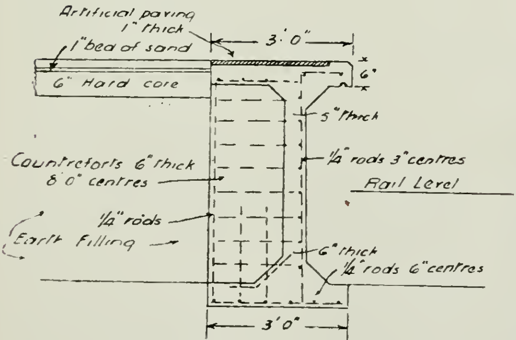
Fig. 4 gives a type of platform suitable for good ground or on arching. In the latter case the concrete would probably be taken down to the backing over the arches, increasing the cost. Here an attempt has been made to cheapen the coping by using fine cement concrete in the place of stone and recessing it, as in *Fig.* 3, to within 6 in. of the chamfer.

This gives a cost of 17s. per sq. yd., as follows :—

	£	s.	d.
1·4 c. yd. Excavation @ 2s.	0	2	9½
1·4 c. yd. Lime concrete @ 12s.	0	16	9½
·5 c. yd. Lime concrete backing @ 13s.	0	6	6
1·4 c. yd. Brickwork @ 24s.	1	11	3
1 c. yd. Brickwork in arch @ 27s.	1	7	0
1·3 c. yd. Filling @ 6d.	0	0	8
6 s. yd. Hard core @ 1s.	0	6	0
6 s. yd. 2½" Flags @ 6s. 6d.	1	19	0
4·5 c. ft. Cement concrete coping @ 1s. 8d.	0	7	6
1·7 s. yd. Pavement cement @ 5s.	0	8	6
	£7	6	0

Area, 12' 0" × 6' 0" = 8 s. yd.
Cost, 17s. per s. yd.

— SECTION. FIG. 5 —



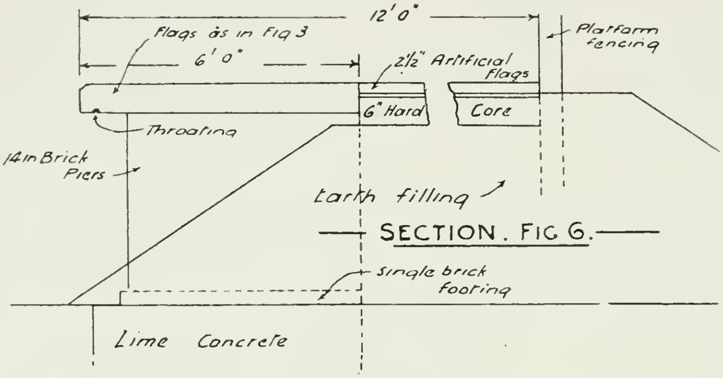
Estimated Cost 16s. 8d per sq. yard.
Wall only 39s. 10d per lin. yard.

The reinforced concrete platform wall shown in *Fig.* 5 costs approximately 39s. 10d. per lin. yd., 6s. 5d. more than for the brick wall in *Fig.* 2, and also more than for an ordinary concrete wall.

The cost of the platform is 16s. 8d. per sq. yd., as follows :—

	£	s.	d.
·5 c. yd. Excavation @ 2s.	0	1	0
19·6 c. yd. Filling @ 8d.	0	13	1
1·7 c. yd. Cement concrete @ 30s.	2	11	0
14 s. yd. Shuttering @ 2s.	1	8	0
·8 cwt. Steel rods @ 15s.	0	12	0
8 s. yd. Hard core @ 1s.	0	8	0
8 s. yd. 2½″ Flags @ 6s. 6d.	2	12	0
2·3 s. yd. Pavement cement @ 5s.	0	11	6
	£8	16	7

Area, 12′ 0″ × 8′ 0″ = 10·66 s. yd. Estimated cost, 16s. 8d. per s. yd. : wall only, 39s. 10d. per lin. yd.



SECTION. FIG 6.

Estimated Cost 13s. 5d per sq: yard.

In *Fig.* 6 is shown a cheaper type of platform.

It is a combination of *Figs.* 2 and 3, and consists of No. 2 8 ft. × 3 ft. reinforced concrete flags supported on 14-in. brick piers, 8 ft. centres. The flags project 12 in. beyond the front face of the piers. The estimated cost is 13s. 5d. per sq. yd., made up as follows :—

	£	s.	d.
·64 c. yd. Excavation @ 2s.	0	1	3
17·5 c. yd. Filling @ 6d.	0	8	9
·64 c. yd. Lime concrete @ 12s.	0	7	8
1·1 c. yd. Brickwork @ 24s.	1	6	5
5·4 s. yd. Hard core @ 1s.	0	5	5
5·4 s. yd. 2½″ Flags @ 6s. 6d.	1	15	1
5·3 s. yd. Reinforced concrete flags @ 6s. 5d.	1	14	3
4·9 s. yd. Pavement cement @ 5s.	1	4	6
	£7	3	4

Area, 12′ 0″ × 8′ 0″ = 10·66 s. yd. Estimated cost, 13s. 5d. per s. yd.

Where the platforms shown in *Figs.* 2 and 6 came in cutting the cost would be slightly reduced, *Fig.* 2 costing 14s. 3d. per sq. yd. and *Fig.* 6 12s. 7d. per sq. yd.

A further reduction would be effected in the case of country platforms by substituting 4 in. of gravel at 9d. per sq. yd. instead of the 2½ in. flags.

PORTLAND CEMENT IN THE RESTORATION OF WINCHESTER CATHEDRAL.

The preservation of our historical buildings requires the closest possible attention, and it is pleasurable to note that Portland Cement, suitably and efficiently applied, may do much to add to the length of life of our important structures in a comparatively economical manner.—ED.

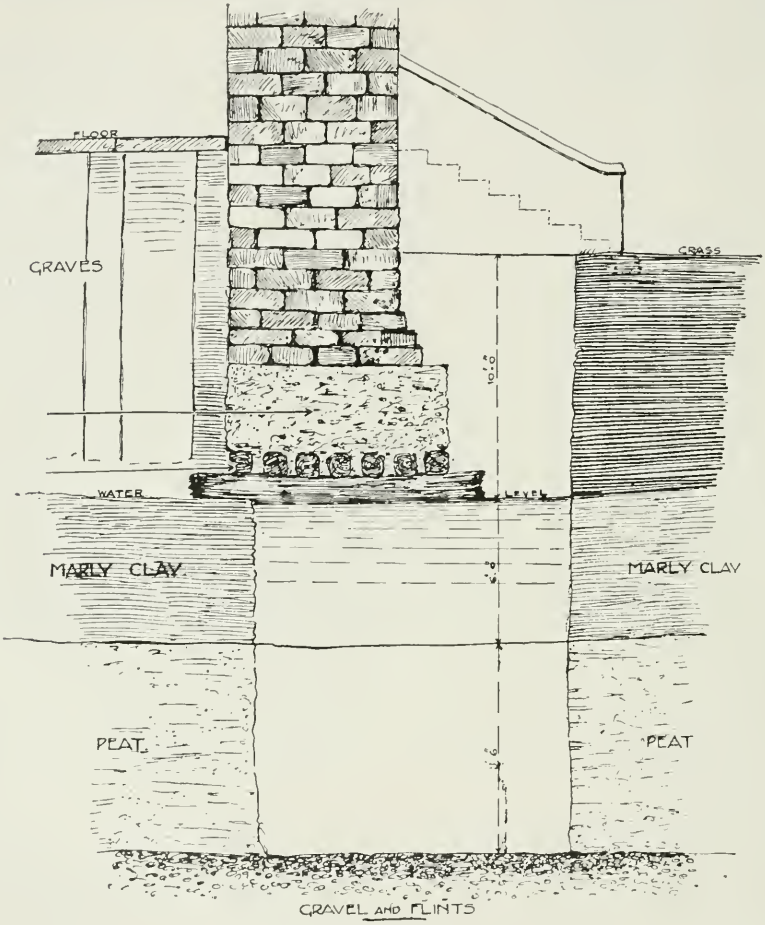
THE restoration of Winchester Cathedral has been dealt with at length in the daily Press, and although no great structural work was entailed, still we think our readers will be interested in this work owing to the fact that cement and concrete played an all-important part in the preservation of this famous building.

The mischief was due to the nature of the foundations, which were not at all adequate for the loamy, watery soil on which the building rests. In Mr. T. G. Jackson's report he mentions that the first part that excited alarm was the south wall of Bishop de Lucy's choir. This had sunk from 7 in. in the middle to nearly 2 feet at the east end and had split off from the Norman work to the west of it, leaving a crack 5 in. wide. It was found that the wall rested on a double layer of beech trees laid crossways, which had been forced down into the soft ground to the extent first mentioned. The north side was not much better, and in both cases the vaults had overpowered the walls and thrust them out of the perpendicular, tearing the buttress piers, on which the thrust was concentrated, away from the curtain walls between them. This movement of the walls had dislocated the stone vaults, which were bulged and cracked in all directions, the ribs having sunk so badly as to have lost their curve of stability, and being only prevented from falling by being bolted to timbering above. The vaulting of the crypts was disturbed throughout, partly by the movement of the walls and partly by the settlement of the piers from which the vaulting sprang.

The great transepts of the original Norman building were in a condition to cause the gravest anxiety. Cracks had appeared in the walls of the north transept so large that daylight could be seen through them, and both the north end and the side walls were dislocated in all directions. But the south transept was far worse, the end wall being broken away by great fissures from the side walls, and having an inclination of over 4 ft. in a height of 120 ft., which nearly brought in the centre of gravity outside the base. The side walls also were badly ruptured, and that on the east inclined seriously outwards.

The problem was how to replace the soft foundation by something solid. By the use of pumps the excavation was continued till a solid ground was reached at a depth of from 16 to 20 ft. below the surface. From this point solid concrete was put in, forming the first "leg" up to the underside of the old footings. Other drifts were formed in the same way, but when a soft, fine silt was found near the gravel, Mr. Francis Fox, M.Inst.C.E., was consulted, and on his advice it was thought better to employ a diver to avoid any danger of disturbing the gravel by pumping.

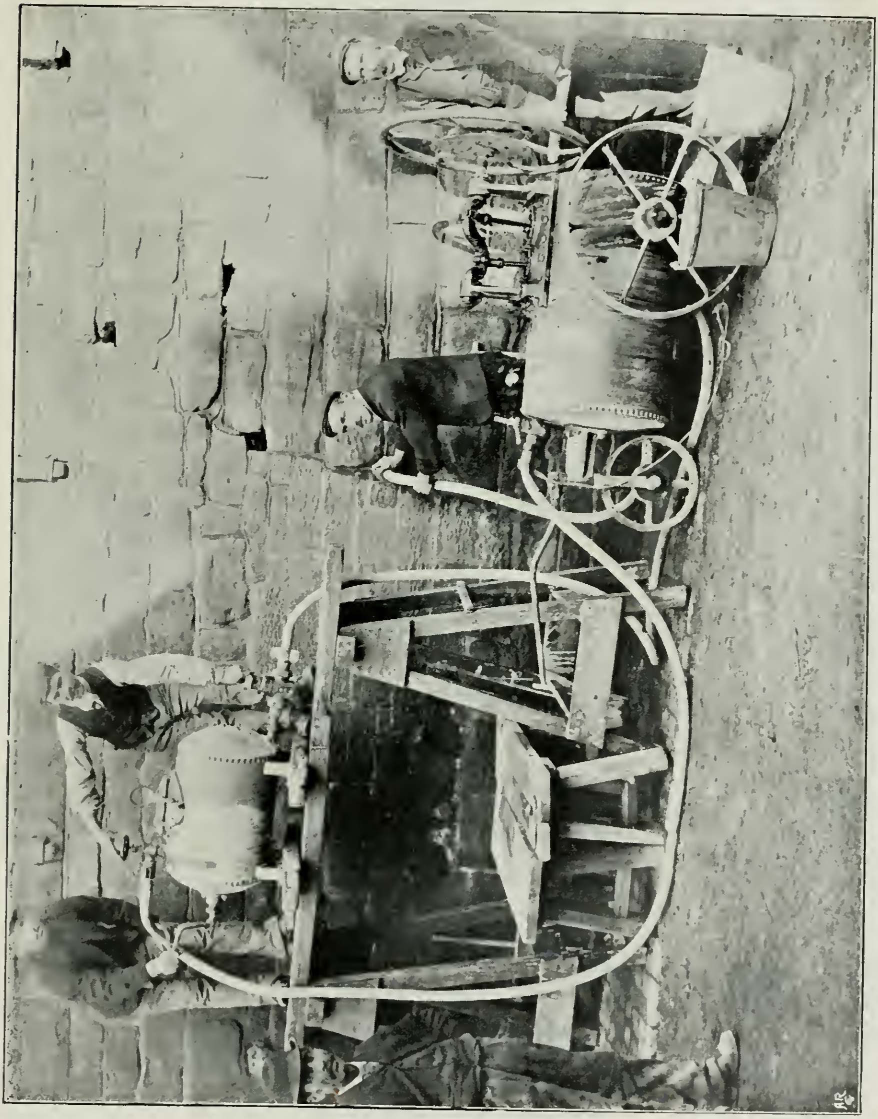
After this the diver was employed all round the cathedral, the modus operandi being as follows:—The wall to be underpinned was divided into sections or drifts about 5 ft. wide. The men excavated each drift down to within about a foot of the gravel bed (pumps going continually all the time).



Section through Wall of Presbytery.
THE RESTORATION OF WINCHESTER CATHEDRAL.

At this stage the pumping ceased and the hole—about 18 ft. long, 5 ft. wide, and from 16 ft. to 20 ft. deep—was allowed to fill with water. The diver then went down and cut out and sent up the remainder of the peat. Bags of cement concrete were lowered to him, and these he spread in the place of the peat, 3 or 4 courses deep, sealing up the gravel. This being done, the hole was again pumped out and the men built up a solid mass with concrete blocks, the topmost 3 ft. being completed with brickwork in cement, pinning up closely under the old walls. This process has been carried out practically round all the external walls, though round the nave the buttresses only have been treated.

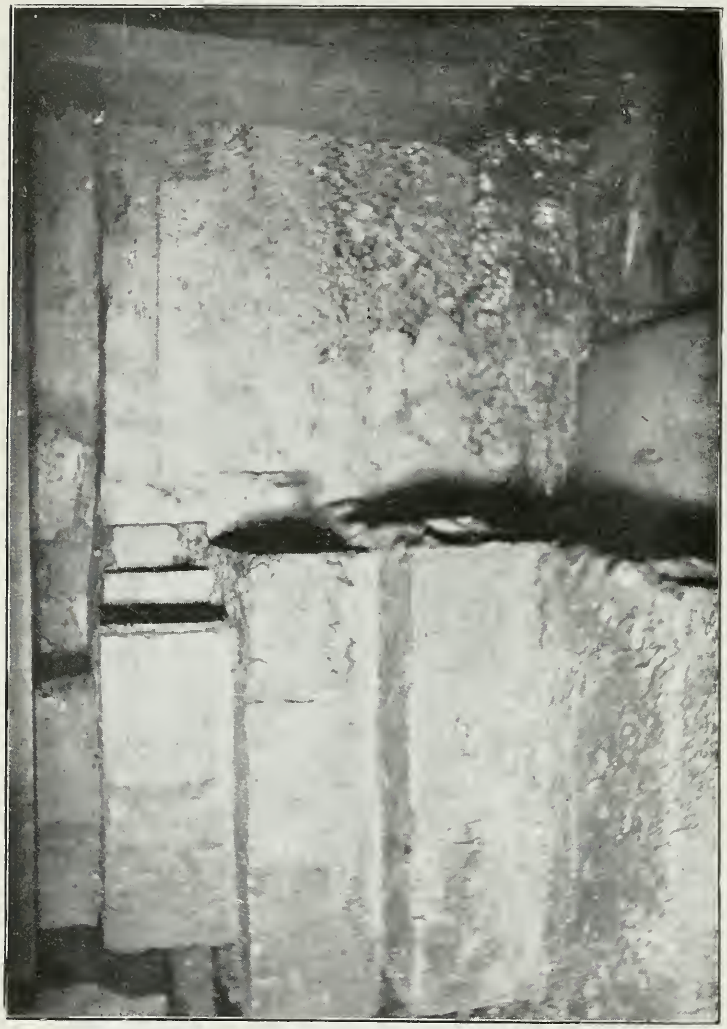
The west front and three western bays were deemed to have sufficient foundations. The piers of nave and transepts, the tower, and the piers of the choir have not been dealt with, as though they rest on exactly the same kind of bottom, the authorities hope that there will be no further sinking of these parts.



Grouting Machine at Work.
The Restoration of Winchester Cathedral.

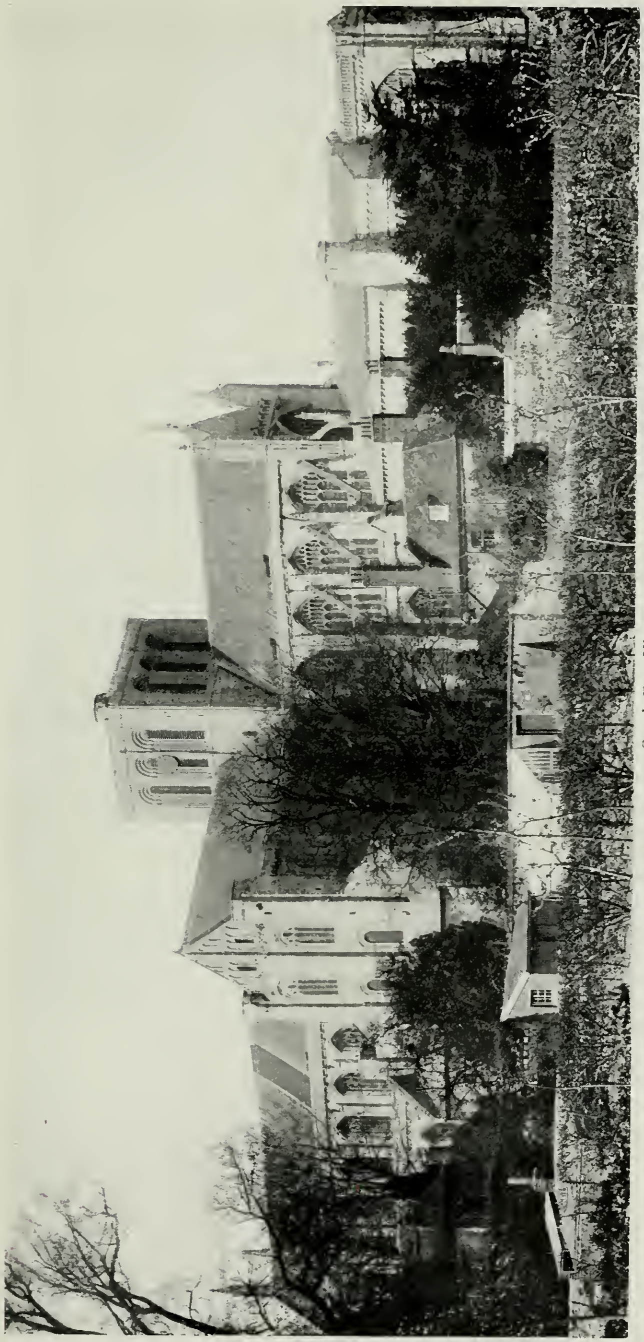
Regarding the cracks of the main walls, of which there were more than 100, large bonding stones have been inserted across the cracks and the walls filled with liquid cement, mostly applied under pressure by the "Greathead" grouting machine. By means of this machine thousands of tons of cement have been pumped into the walls, travelling in every direction throughout the masonry, 25 tons of cement having been forced into one corner of the north transept alone.

In a paper read before the Royal Institute of British Architects in 1908, entitled "Foundations," Mr. (now Sir) Francis Fox described the work at Winchester Cathedral and the importance of the grouting machine for this work. The accompanying illustrations, which were kindly placed at our disposal by the Royal Institute of British Architects, were included in that paper. The machine consists of an iron receiver or reservoir, into which, by means of pumps, air can be forced under any pressure up to 100 lb. to the inch. This receiver is connected by a flexible tube to another portion of the apparatus, called the "grouting pan," which is, in fact, a churn furnished with a handle and spindle, to which are attached arms or beaters. The proper proportions of cement and water, and in certain cases sand, are then placed inside, the lid screwed down, and the contents churned up into the consistency of cream. This is now ready to be blown into the crack, the mouth of which on either side of the wall has meanwhile been clayed up to prevent the grout from escaping. The compressed air is then admitted to the grouting pan, and so soon as the necessary valve is opened the contents are discharged into the wall.



View Showing the Concrete Bags Deposited by Diver.
THE RESTORATION OF WINCHESTER CATHEDRAL

By means of this apparatus cement can be blown right into the heart of any structure, whereby all the loose particles of stone and the opposite sides of the crack can be agglutinated, or, more properly, cemented together, and we have thus the power of repairing injured buildings without being compelled to pull them down.



South-East View.
The Restoration of Winchester Cathedral.

Where the groining or vaults had sunk or broken, it has been replaced in its original position, and all grouted up with cement and made solid. Steel ties have been used in various places. In the south transept three buttresses have been built, and the south wall of the nave has been supported by flying buttresses.

The greatest credit is due to Mr. T. G. Jackson, R.A., the diocesan architect, and to Sir Francis Fox, for the work they have done in preserving one of the most beautiful buildings in the country. It may be said that the entire cathedral has been welded together and placed upon a new foundation, and it is expected that the structure will stand for many generations.

The contractors for the whole of the work were Messrs. Thompson, of Peterborough.

REINFORCED CONCRETE ARCH BRIDGE DESIGN.

By MR. DANIEL B. LUTEN, Member of the Western Society of Engineers.

A very interesting paper on Arch Design was read before the Western Society of Engineers (of U.S.A.) in June last by Mr. Daniel B. Luten, from which we take the particulars below. The paper was exceedingly well illustrated by views of bridges designed by Mr. Luten, a few of which we reproduce herewith.—ED.

TWENTY years ago there was hardly a concrete arch in the United States; ten years later there were less than a hundred; to-day there are upwards of ten thousand. This suddenly accelerated growth is due to a realisation that the concrete arch fulfils most of the requirements of an ideal highway bridge. Arches of material other than concrete, such as stone, brick, and steel, have been lacking in some important attribute, so that no sufficient number have been erected in a limited period to enable any engineer to specialise exclusively in arch design in those materials.

Requirements of the Ideal Highway Bridge.—The ideal highway bridge must include among its qualifications the following :—

1. Permanence, eliminating repairs and inspection.
2. Artistic appearance to harmonise with its surroundings.
3. Strength increasing with time and traffic.
4. Safety, meaning not merely security, but slow failure in case of defects.
5. Stable on insufficient foundations and under extreme flood conditions.
6. Of effective waterway providing maximum discharge.
7. Efficient and economical in use of materials.
8. Employing home labour and materials.
9. Providing a roadway continuous over bridge and approaches.
10. Easily widened to provide for increasing traffic.
11. Easily modified in design to conform to improvement in surroundings.
12. Simplicity in design and erection.

HOW THE REINFORCED CONCRETE ARCH COMPLIES WITH ABOVE REQUIREMENTS.

Permanence.—An arch of concrete reinforced with embedded steel has all the permanence of stone; in fact, it is more permanent than the usual building stones, and has none of the limitations of steel, such as corrosion and crystallisation; for concrete is but slightly affected by the elements, and the embedded steel is protected from rust and vibration. No painting or repairs are ordinarily required, and inspection is superfluous.

Artistic Appearance.—A properly-designed reinforced concrete arch will be in equilibrium under the fixed load. Reinforcement will be required in the arch ring to resist moving load stresses only, and the moving loads are small in comparison with the fixed load. Such a structure, therefore, displays its material in an efficient form, and because of its useful application presents a beautiful appearance. Everyone knows that concrete is a material strong in compression. Its use in the arch without ornamentation consequently appeals to the eye. This is wholly apart from its relation to its surroundings, in which harmony must be secured ordinarily by ornamentation of spandrels and railings. There is no harmony, for example, in adopting a segmental curve or parabolic arc for an earth-filled arch merely because it carries railway loading, when such curves are not the curves of equilibrium, for the loading.

Since the concrete arch is of pleasing form when properly designed, all that is necessary to make it harmonise with its surroundings is to limit the design to dignified details for rugged surroundings, and to embellish it with ornamentation for cultivated surroundings. No other bridge structure harmonises so readily with its surroundings, no matter what they may be. The mechanical lines of the truss and girder appeal to the eye of the trained engineer and are usually more pleasing on the drafting table and in the blue print than the arch. But the actual structure after erection and in perspective is nearly always a disappointment both to the engineer and the layman when compared with a well-designed arch.

For spans of less than 100 ft., dignity of design can usually be best secured for an arch bridge *of moderate rise* by the use of the earth-filled type with solid spandrels. But a semi-circular arch can hardly be made to present a satisfactory appearance with solid spandrels, unless the depth of load over the crown be proportional to the rise, not an economical arrangement. For spans greater than 100 ft., especially with great rise, the open-spandrel construction is usually to be preferred, and this accords fairly well with economy in this country at the present cost for labour and materials. In Europe, where labour is relatively cheaper, it may be desirable from the standpoint of economy to use open spandrels on spans as short as 60 ft. or 75 ft. A hollow spandrel behind a solid spandrel wall is rarely desirable, for the curve is then not in harmony with its loading except for very flattened spans. In a concrete girder it is extremely difficult to show by external treatment the presence of the steel that resists one-half of the stresses. Appropriate panelling will accomplish it in part, but in any event the reinforced concrete girder must always leave an impression of weakness that cannot be satisfied by any external treatment.

An arch bridge, whether in single span or in series of arches, presents the best appearance when cambered in a vertical parabolic curve having a rise of about 1-150 the entire length of the bridge, but not ordinarily to exceed 3 ft. This ornamental camber in the arch bridge also provides for surface drainage of the roadway, which may be discharged through the crown of each arch, if not objectionable below.

Increasing Strength.—Timber, steel, and even stone deteriorate as they grow older. Concrete grows stronger with age. The increase may amount to 50 per cent. in the year following the first thirty days. Since traffic on most of our highways is constantly increasing in weight, such an increase in strength tends to keep pace with the growth of traffic. A bridge that decreases in strength must be built excessively heavy, for the day soon comes when its decreasing strength meets the increasing traffic, and collapse results. The bridge that increases in strength with age may be designed for efficiency. Hence the light efficient design is to be commended in such a structure, whereas it receives merited condemnation in steel or wood.

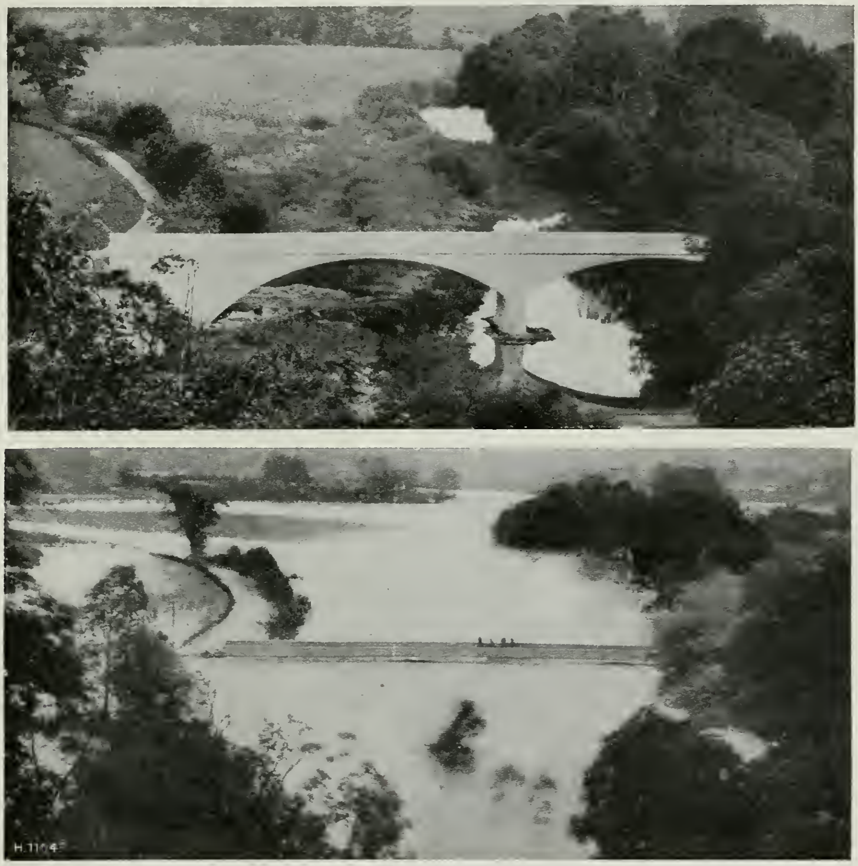
The concrete arch, since it depends for its strength principally upon the compressive strength of the concrete, does increase in strength as the concrete gains in strength, and it is the only type of bridge that possesses this property. The concrete girder, designed for equal efficiency of steel and concrete, cannot gain any advantage from the increasing strength of the concrete except by overstressing the steel. Designing the girder with excessive allowable stress for concrete in compression enables the girder to grow in strength with age, for a limited time only, but at the risk of a collapse in the early period of its existence. This is taking the chance that no full load comes over the structure in its first year. As the concrete of the arch hardens the elastic arch becomes less flexible and will distort less under a concentrated load. Hence the tension in its embedded steel will be reduced as the structure grows older. And since the concrete grows harder and better able to resist higher compressive stresses, while the tension in the steel is at the same time reduced, the structure as a whole grows stronger as its age increases.

Reinforced Concrete Bridge with an 80-ft. span, at Ripley, New York.

Span of Waterway, 65 ft.
Reinforced Concrete Bridge at Racine, Wisconsin.

An earth-covered arch also increases considerably in strength but in an unknown ratio, by reason of the compacting of the earth fill over the arch and behind the abutments. Many earth-covered arches have been erected with light abutments reinforced with ties from abutment to abutment to resist the thrust, from which the ties have been removed after two or three years to permit the deepening of the channel, and with no resultant yielding of the abutments.

Safety.—A bridge made of any structural material may be designed to safely support its load by a proper distribution of the material. To make a bridge secure, therefore, whatever the material or type, is simply a matter of design. But no matter how well planned, a bridge may be defective in erection. In such a case a bridge that fails slowly and with abundant warning is superior to one that collapses suddenly. A structure in which tension forces are isolated in separate members, as in a truss, fails suddenly when any member gives way. A girder of steel or concrete in which the dead load forces are concentrated in extreme regions, as top and bottom flange, will follow the same behaviour in a less marked degree. A concrete arch with open spandrels supported by separate arch ribs will collapse suddenly in limited regions supported by a defective rib. But a concrete arch covered with earth filling and stiffened with spandrel walls not only fails slowly but gives abundant warning of overstress. Failure in such a structure is almost always by yielding of abutments or piers



View Showing Behaviour of Arches in Excessive Flood
REINFORCED CONCRETE BRIDGE AT SALEM, INDIANA.

which move slowly. There are many instances of reinforced concrete arches which have settled continuously for months, developing cracks but without failure. In one instance the crown settlement in an 80-ft. span amounted to several inches before it was stopped by a support under the crown.

Reinforced concrete has one marked advantage over most other materials, in that it may indicate conditions of maximum allowable tension in its embedded steel before actual danger exists. This advantage rests in the fact that the coefficient of elasticity of the concrete and of the embedded steel do not bear the same ratio as their allowable stresses. When the embedded steel is stressed to 5,000 lb. per sq. in. invisible cracks occur in the surrounding concrete. At 10,000 to 15,000 lb. per sq. in. these cracks become visible. At 20,000 lb. per sq. in. they frequently become objectionable.

In arch design, where the stresses in the arch are magnified by the behaviour of the spandrel walls, cracking of the concrete serves the purpose of an extensometer to detect excessive stresses. If an arch is designed too flat at the crown cracks will appear in the spandrel near the ends of the span. If too flat at the haunches, cracks will appear in the coping over the crown or through the arch ring at the haunches directly under the spandrel only, and not extending far into the soffit of the arch. The tensile stress in the arch itself is rarely sufficient to show cracks actually penetrating the arch ring. Small cracks, particularly in the spandrels, are no indications of failure, being merely the magnified effects of movements in the arch ring, but a *properly designed and erected* arch will be free from such cracks, if provided with expansion joints in the spandrels above each springing.

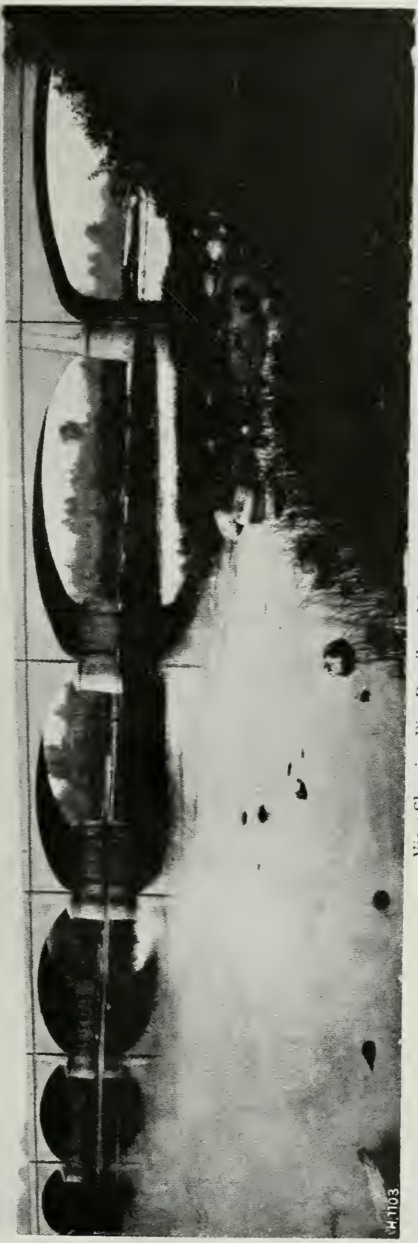
Stability — The arch imposes a heavier load upon its foundations than any other type of bridge. But since there is no limit to the expansion possible in footings, the intensity of pressure may be made the same as for any other structure by proper distribution, so that the heavier loading of the arch is objectionable only as affecting its comparative cost. It proves a decided advantage, in fact, when the effects of moving loads on the structure are considered. The heaviest locomotive loading may increase the foundation pressures under a steel trestle of 100 ft. span several hundred per cent., whereas the corresponding increase under an earth-covered arch of the same span rarely exceeds 20 per cent. The stresses in the latter structure itself are not seriously affected by a considerable increase in the moving load.

Steel and wooden bridges have one great advantage over concrete bridges in stability, in that they can sustain unequal settlements without serious damage. Concrete bridges are not seriously affected by uniform settlement of one or both abutments. But unfortunately, settlement of an abutment is very rarely uniform. Unless founded on rock, the conditions underlying any abutment of 18 ft. or 20 ft. length in the average stream are of varying character, whether clay, sand, gravel, or piling. Settlement, if it occurs, due, of course, to excessive intensity of pressure, will always be greater at one point than another. If an abutment settled uniformly it would not crack. But how many abutments that have settled are free from cracks?

The concrete arch with its great mass is not readily moved by floods nor ice jams. It is consequently a desirable type of structure for a low-level bridge subject to complete submergence. In such cases its roadway and approaches should be paved to prevent scour. The wide abutments required by a concrete arch to resist its horizontal thrust add to its stability in flood, for such an abutment may often be undermined for a considerable area at the stream face without injury, since the maximum pressure is at the back.

Waterway. — For the purpose of comparison, consider the three fundamental types of waterway possible, V-shaped, rectangular, and inverted V, all of equal area. The V opening is the type desirable for sewers where a small flow of water must produce a deep swift current to carry solids along a uniform channel. It is not suitable for an opening for a bridge, since it must dam back the water before capacity can be secured. It requires the longest span for a given area of waterway.

The inverted V opening provides the greatest span at the lowest level, hence it will provide the greatest discharge at low levels and with least damming effect. It requires a long span, but permits the use of the arch instead of the girder.



View Showing Pier Details and Long Spans on Light Piers.
Reinforced Concrete Bridge, Wayne Street, Peru, Indiana.

The rectangular opening is intermediate to the other two in effective discharge, and has the advantage of shorter span, limited, however, to the girder type. It appeals to most designers because the entire superstructure can be made to clear the former high-water records. But this apparent advantage rests wholly on the assumption that future high-water levels will conform to that record.

For the same waterway area, therefore, the arch type of opening has a considerable advantage over the girder and sewer forms. But for a given span and a fixed water level this advantage no longer obtains.

Efficiency.—Concrete is a material high in compression values and low in tension and shear. If an arch be designed as the linear arch for the fixed load, it will have no tension, and no shear whatever except in very slight degree for temperature changes and moving loads. The stress in such an arch approximates very nearly uniform compression throughout. Consequently, for a location where rock foundation is exposed near the surface, no structure of concrete can compete in cost with the arch. But when footings must be expanded to provide for soft foundations or insufficient backing, the advantage is less marked. Nevertheless, the reinforced concrete arch is usually of lower first cost than the concrete girder for the same requirements, in spans exceeding 20 ft. or 30 ft. It will also compete with steel girders and trusses having permanent floor and substructure.

Continuous and Widened Roadway.—A bridge that requires an earth covering has an advantage in that the earth roadway, or gravel, macadam or brick pavement may be extended directly across the structure with no break whatever in its continuity. It is objectionable to build with a level floor of concrete, or any other material that does not have provision for carrying the same materials used in surfacing the highway leading to the bridge. For a change in paving material usually means a chuck-hole where the harder pavement joins the softer. And even an earth covering over a concrete deck is apt to show such depressions after wet weather, unless the earth or gravel covering is more than 12 in. thick. The covering over an arch reduces in depth gradually to a desirable minimum of 8 in. to 12 in., and there is, therefore, no formation of chuck-holes if earth covering is used, nor any difference between the earth covering of the bridge and the earth foundations of the road or street itself. Any kind of pavement or surface can thus be obtained over the arch with the same construction as the roadway approaches.

As a community grows older it frequently becomes necessary to widen bridges to provide for increasing traffic that could not have been foreseen, or that would have required excessive cost in the original construction. It sometimes occurs that a bridge is erected with a 16-ft. roadway, and that later the coming of an inter-urban railway builds up the community about the bridge, necessitating a structure of much wider roadway. Through bridges of truss or girder type cannot ordinarily be widened without scrapping the superstructure. If the supporting steel trusses or girders were originally heavy enough to carry the additional load, which, of course, is rarely the case, they can be moved apart and supplied with heavier floor beams.

Deck bridges and arches can be widened at any time without loss of any of the original investment except moving the railing and possibly burying the spandrel walls in the added fill.

Simplicity in Design and Erection.—In the matter of analysis, the reinforced concrete arch is far from being ideal and is perhaps as difficult an engineering structure to deal with as has been devised.

In erection, on the contrary, simplicity lies with the arch. The erection of centres and placing of reinforcement requires only the ordinary skill of an average carpenter. The finishing of the concrete requires skilled men, but a larger proportion of the concrete arch is erected with unskilled labour than in the case of any other type of bridge. A concrete girder is more simple in form erection than the arch, but much more complicated in the placing of the steel reinforcement, which is not only four or five times as heavy as that for the same span arch but is also much more complex in details of fabrication.

LIMITATIONS OF THE ELASTIC THEORY.

The elastic theory as applied to reinforced concrete arches is still in its elementary stages of development. No method has yet been devised of satisfactorily analysing an arch of concrete in which any material tension occurs. No satisfactory method of determining the tension stress in the steel is known. No text-book shows any proper application for moving load analysis for arches on piers. A serious objection to the elastic theory is that as ordinarily applied it shows excessive stresses at the springings for temperature changes. These extreme stresses do not actually occur, and some designers have endeavoured to offset them by permitting higher allowable stresses for temperature variations or by assuming a low range of temperature. Just why a higher allowable stress should be permissible for temperature variations, which recur more frequently than for a fixed load, is difficult to explain. Common sense would indicate, regardless of experiments, that a range of ±40 deg. would be the least that could be expected in this latitude.

30 ft. Span.

Reinforced Concrete Bridge in Columbia Park, Lafayette, Indiana

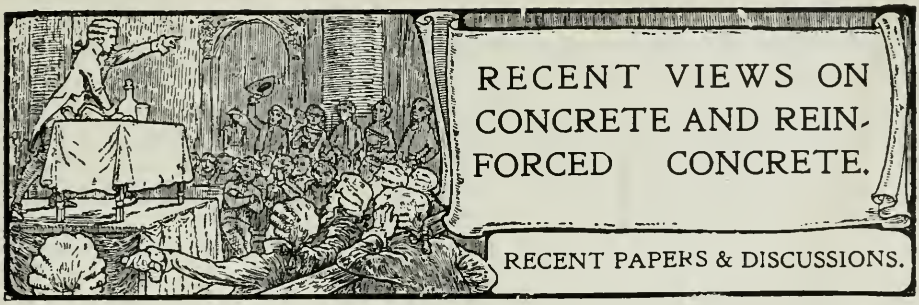
Even so good an authority as Professor Howe says in his text-book on "Symmetrical Masonry Arches," page 81 :

> "As a large number of railway bridges have been built upon practically the dimensions we assumed, and no indications of failure having been found, we must conclude that the range of temperature change (±40°) assumed in this example is very much too great. Furthermore, it requires a drop in temperature of only *four* degrees to completely balance the compression produced by the dead load in

> the upper fibres at the support. Without question, then, our assumptions about the effect of temperature changes are not correct. Until we know more about the subject it is useless to make calculations according to the ordinary assumptions."

Yet the range of temperature undoubtedly *is* as great as that assumed, and this has been confirmed by recent experiments. A large part of the difficulty consists in the limited length of arch ring usually assumed to be elastic.

The temperature stresses in an arch are not as extreme as those in the bedstead girder type of concrete bridge, in which the span is supported on legs of reinforced concrete with no provision for expansion and contraction. For spans exceeding 30 ft. the temperature stresses in such a structure may become very severe, but are often overlooked because the girder and abutment are analysed separately, whereas the elastic theory for arches necessarily involves the relation of the abutment to the arch. The detrimental effects of temperature in a bedstead concrete bridge are magnified, too, by the earth pressure back of the supports.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE CONCRETE INSTITUTE.

INTERIM REPORT OF THE TESTS STANDING COMMITTEE ON THE TESTING OF CONCRETE, REINFORCED CONCRETE, AND MATERIALS EMPLOYED THEREIN.

The interim report of the Tests Standing Committee of the Concrete Institute on the Testing of Concrete, etc., was discussed at a meeting of the Institute held in April last, and we give a summary of this discussion below. The report itself had already appeared in our issue of July, 1911, *but we publish it again so that our readers may study it in conjunction with the discussion.*

For the purpose of this report the materials employed in concrete and reinforced concrete will be classified under the following headings :—

(*a*) Coarse Material.
(*b*) Sand.
(*c*) Cement.
(*d*) Water.
(*e*) Steel.

This report does not take into consideration, so far as applied to reinforced concrete, any coarse material which does not pass an aperture of $\frac{3}{4}$ in. by $\frac{3}{4}$ in. and which is not retained on an aperture of $\frac{1}{4}$ in. by $\frac{1}{4}$ in. The material used for coarse material generally can be classified under the following headings :—

(*a*) Pit gravel.
(*b*) Do. crushed.
(*c*) River gravel.
(*d*) Do. crushed.
(*e*) Sea gravel.
(*f*) Sea gravel. crushed.
(*g*) Crushed rock.
(*h*) Crushed brick.
(*i*) Crushed slag
(*j*) Clinker and coke breeze.

Classes (*i*) and (*j*) are not recommended as suitable material for reinforced concrete.

This Committee is of opinion that to obtain the best results in concrete, the coarse material and sand should be properly graded.

Unfortunately the principle of grading coarse material and sand is very little resorted to, and consequently there are few data on this important subject.

This is now being considered in detail by the Committee, and a further report will be issued in due course.

The accompanying diagram giving the result of a series of tests shows that the percentage of voids in materials varies according to the different sizes and grading from 35 per cent. to 48 per cent. It should be noted that, owing to the different materials tested, the curve is very irregular.

This report does not take into consideration, so far as applied to reinforced concrete, any sand which does not pass an aperture of $\frac{1}{4}$ in. by $\frac{1}{4}$ in., and which is not retained on an aperture of $\frac{1}{50}$ in. by $\frac{1}{50}$ in.

Sand used in concrete work is as follows :—

(*a*) Pit sand (other than that of glacial origin).
(*b*) River sand.
(*c*) Sea sand.
(*d*) Grit or sand from crushed coarse material.

It is important that all sand that is not retained on an aperture of $\frac{1}{50}$ in. by $\frac{1}{50}$ in. should be rejected. The following results of tests have been submitted to this Committee as showing the deleterious effect of allowing dust to remain in the sand :—

PROPORTION. THREE TO ONE (BY WEIGHT).

Crushing Tests.

British Standard Sand and Cement. Per Square Inch. 28 Days.	Crushings from Porphyritic Granite and Blue-grey Whinstone to pass 30 x 30 Sieve, including all Dust. Per Square Inch. 28 Days.
2,789 lb.	606 lb.

Tensile Tests.

7 Days.	28 Days.	7 Days.	28 Days.
182·2 lb.	234·6 lb.	53·2 lb.	104 lb.

Average of Five Tests at each Period.[1]

The percentage of voids in sand (which varies from 23 per cent. to 40 per cent.) is shown in the accompanying diagram.

As will be seen from the accompanying diagram, according to the various proportions of graded coarse material and sand, the percentage of voids varies from 24 per cent. to 30 per cent.

All cement should be tested and analysed in accordance with the Engineering Standard Committee's latest specification. The Tests Standing Committee of the Concrete Institute strongly recommends that in addition the crushing strength of the cement should be ascertained in the following manner :—

Cubes mixed in the proportion of 3 parts of standard sand to 1 part of cement, by weight, shall be made up and treated in all respects similar to briquettes of sand and cement for the tensile test, but the test specimens for crushing to be 3-in. cubes. The crushing strength shall be not less than ten times the tensile strength at twenty-eight days required by the British Standard Specification.

This Committee is of opinion that it is not advisable to define the proportion of water to be used in making up concrete or test specimens of cement and concrete, as this quantity must be more or less dependent on the variations in the weather and the nature of the coarse material and sand used in each particular case; but it is generally considered that the concrete should be mixed to a plastic state, though not so wet as to allow any dripping of the cement, water, and sand.

The following tests are recommended for all steel work for use in reinforced concrete.

The steel shall attain an ultimate tensile strength of not less than 60,000 lb. per sq. in.

The steel shall withstand a stress of at least 34,000 lb. per sq. in. before showing any appreciable permanent set.

The contraction of area at fracture shall be not less than 45 per cent.

(Or) The elongation in the case of bars of 1-in. diameter and under shall be not less than 25 per cent. measured on a length equal to eight times the diameter of the bar tested.

The elongation shall be measured in the case of bars over 1-in. diameter on a length equal to four diameters of the bar, and shall be not less than 30 per cent.

All steel shall stand bending cold to an angle of 180° around a diameter equal to that of the piece tested, without fracturing the skin of the bent portion.

The steel shall be free from scabs and flaws.

For the purpose of ascertaining the crushing resistance of concrete the following particulars shall be complied with :—

(*a*) All test pieces shall be 4-in. or 6-in. cube.
(*b*) Number of test specimens for each test shall be six.
(*c*) Three cubes shall be made in the laboratory and three for each test shall be made on the works.

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[1] Tests made for Messrs. Scott & Fraser by Messrs. D. Kirkaldy & Son.

(*d*) The cement for each series of cubes shall be taken from the same consignment.

(*e*) All laboratory-made test cubes shall be made as far as possible on practical lines, so that the results of the tests shall be such as can be reasonably expected from concrete in actual work with proper care and attention.

(*f*) Specimen cubes from the works shall be made with concrete taken from the actual mixing platform; the concrete used for the specimen cubes shall be taken from the various batches of concrete prepared for use in the works.

(*g*) In all cases specimen pieces shall be made in metal moulds, and the concrete worked in by "punning" and "tamping" and afterwards gently rammed.

The following particulars shall be taken or ascertained of the materials used in the test specimens of the concrete:—

(*a*) Full test on cement as specified in Clauses 12 and 13.

(*b*) The coarse material and sand for the laboratory tests, which is to be taken from bulk as delivered to the works, shall be sieved to the following degrees and the voids ascertained of (1) the whole and (2) of each separate grading:—

Coarse Material.

To pass an aperture of—	To be retained on an aperture of—
$\frac{3}{4}$ in. by $\frac{3}{4}$ in.	$\frac{5}{8}$ in. by $\frac{5}{8}$ in.
$\frac{5}{8}$ in. by $\frac{5}{8}$ in.	$\frac{1}{2}$ in. by $\frac{1}{2}$ in.
$\frac{1}{2}$ in. by $\frac{1}{2}$ in.	$\frac{3}{8}$ in. by $\frac{3}{8}$ in.
$\frac{3}{8}$ in. by $\frac{3}{8}$ in.	$\frac{1}{4}$ in. by $\frac{1}{4}$ in.

Sand.

To pass an aperture of—	To be retained on an aperture of—
$\frac{1}{4}$ in. by $\frac{1}{4}$ in.	$\frac{1}{8}$ in. by $\frac{1}{8}$ in.
$\frac{1}{8}$ in. by $\frac{1}{8}$ in.	$\frac{1}{16}$ in. by $\frac{1}{16}$ in.
$\frac{1}{16}$ in. by $\frac{1}{16}$ in.	$\frac{1}{32}$ in. by $\frac{1}{32}$ in.
$\frac{1}{32}$ in. by $\frac{1}{32}$ in.	$\frac{1}{50}$ in. by $\frac{1}{50}$ in.

(*c*) The proportion of each grading to the whole

(*e*) The specific gravity of the coarse material and sand.

(*b*) The exact dimensions of specimen cubes.

(*c*) The weight per cubic foot of all specimens immediately before testing.

(*f*) The testing of concrete specimens shall be conducted on three laboratory specimens, and three specimens made on the work (see Clause 21), and shall be tested at the following periods:—

Minimum Tests.	7 days
(Recommended as the least that should be undertaken).	28 ,,
	56 ,,
	90 ,,
	6 months
	12 ,,

Medium Tests.	7 days
(Recommended to be undertaken wherever possible).	28 ,,
	56 ,,
	90 ,,
	9 months
	1 year
	2 years

Maximum Tests.	7 days	1 year
(Recommended for extensive structures and research).	28 ,,	2 years
	56 ,,	3 ,,
	90 ,,	4 ,,
	6 months	5 ,,
	9 ,,	

All specimens shall be kept in air after mixing and slightly damped for the first seven days.

All cubes made up on the works shall be forwarded to the laboratory on the fifth day.

After the expiration of seven days specimens shall be kept under cover for the purpose only of protection from the direct action of rain and direct sunrays.

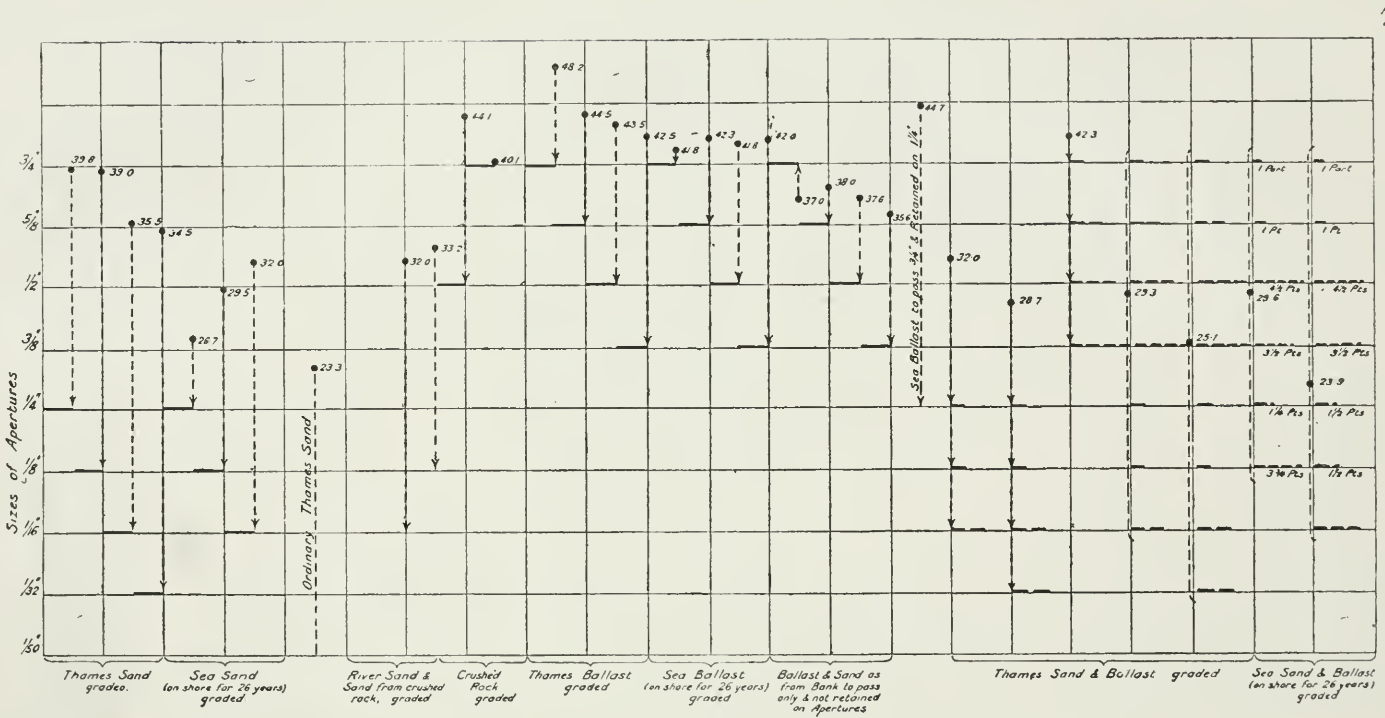


DIAGRAM SHOWING PERCENTAGES OF VOIDS IN VARIOUS BALLASTS AND SANDS.

— *EXPLANATION.* — *Each Horizontal Thick Line represents the Percentage of parts which pass the size Aperture shown opposite the Line and which is Retained on the Size Lower (except No 8)*

The Figures shown Represent the Amounts of Voids

The Committee is of opinion that for the purpose of providing for the cost of testing a provisional sum should be included in all contracts where such testing will be required, this being the most satisfactory and fairest way to all parties concerned.

(Signed) WILLIAM G. KIRKALDY,
Chairman of Tests Standing Committee.
(Signed) A. ALBAN H. SCOTT,
Hon. Secretary of Tests Standing Committee.

REPORT OF THE TESTS STANDING COMMITTEE ON THE TESTING OF REINFORCED CONCRETE STRUCTURES ON COMPLETION.

IT is frequently specified that test loads should be applied to finished structures of reinforced concrete shortly after completion. It should be recognised, however, that such tests should in no wise reduce the care to be exercised in the supervision of the work by those responsible. Since the test loads are generally applied only to specific parts of the work, such tests do not necessarily prove that the work has been properly executed either in whole or in part.

The test load should not be applied to any part until the expiry of ninety days from the last day of laying the concrete.

The deflection of beams under a test equal to the full working load for which the beams were designed should not exceed 1/1000th of the span.

In order to impose the full load upon a floor or roof beam under test the two bays of floor or roof adjoining require to be loaded all over, otherwise a considerable portion of the load is transmitted to the adjoining beams and the full load does not come upon the beam under test.

Not more than the superimposed load for which the beam has been designed, plus 50 per cent., should be applied as a test load.

When test loads are applied the materials used for loading should be put on in such a manner that no arching action whatever can take place, otherwise incorrect results will be obtained.

If systematic and thorough supervision be given by the professional adviser during course of construction the application of test loads to the finished structure is not so necessary.

Test cubes, if kept in an even temperature, which should be the same as that specified for cement briquettes, form a means of gauging under standard conditions the resistance to thrusting, and cubes kept in the open air will give the values for the concrete in actual practice. The results of these two sets of tests will be helpful in determining at what age the work is strong enough to sustain the loads.

DISCUSSION.

Mr. M. E. Yeatman, M.Inst.C.E., M.Am.Soc.C.E. — He had not had an opportunity of making a study of the report till he came into the room, and had not gone sufficiently into the question of testing. There were many points, however, which it would be valuable to have discussion upon.

The question of the proportion of water had always been a difficulty, and he thought the Committee were probably wise in putting it, that it was impossible to prescribe numerically the proportion of water to be used. It must be judged by the conditions and the results in mixing.

In the question of the grading of the materials it was very valuable to have the materials measured or tested in the manner prescribed, to see what proportion there was of all the different sizes and what proportion of voids each size left. But in his opinion it was not desirable to use materials of uniform sizes. The different proportions should be mixed, for this reason, namely, that a uniform material, that is to say a material in which all the particles were about the same size. would leave an amount of voids ranging from 30 to 50 per cent., or 45 per cent., sometimes it is nearly 50 per cent., so that you could not secure having the voids in that broken stone filled unless you had at least half its amount of sand And you could not secure the voids in that sand being filled by cement unless you had at least half the amount of the cement, that is, if the broken stone and the sand were respectively of uniform size. But if the sand consisted of particles of a good many different sizes, then the smaller particles would fill up some of the voids between the smaller ones, and it would not require so much cement to fill up the ultimate ones, in fact, it would be found that in material where the particles consist of a lot of different sizes mixed the total number of voids would be a good deal less than if all the sizes were uniform. In describing mixed concrete that fact should be taken into account, and in order to know how to deal with your materials it was very valuable to know of what different sizes they were composed, and in what proportions those sizes were present.

It was very wise to specify the time after the conclusion of the work before a test could be safely applied. He had heard of tests being applied a very short time after the work was set, which was very risky, though ultimately the strength might be all that was needed. Ninety days was, he thought, a wise provision.

Mr. William G. Kirkaldy, Assoc.M.Inst.C.E. — He agreed with Mr. Yeatman that the sand should have as many different sizes as possible, and should have as few voids as possible. The idea of taking the voids separately was really to find out whether there was a preponderance of large grade in the sand or a preponderance of fine stuff, but the idea was that we should not have uniform sand, but to attack the question to find out what the proportions were, and then to give a chance to add any more of a certain size, if possible, to make up any deficiency. If the sand ran too fine, with no coarse grade, there would be a chance to correct that. Certainly it was very desirable to get the size as varied as possible from the coarse down to the fine. That makes a denser concrete, and, of course, better work and a much higher physical strength too. The crushing strength was in very direct relation to the sands in the concrete. And, of course, it was economy in the cement as well. If you could get your aggregate in the sand to have as few voids as possible the cement was practically easy for you. It was wasteful to use cement for filling cavities, as used to be done in the old days, before they studied concrete questions as they had recently.

He could assure the meeting that very great consideration was given to the report by the members of the Committee, so as to try and cover all the ground. Of course, they aimed high. Some men might think the tests are more complete than necessary, but they could always be cut down. The tendency in practice was to have fewer tests, but they had started out to give a programme of what they thought would be advisable.

Mr. D. B. Butler, Assoc.M.Inst.C.E., F.C.S. — Mr. Kirkaldy made one point as to the tests of concrete and suggested their being fewer; perhaps they would be almost cleared out. Well, he thought there was a good deal in that point because if all these tests which are mentioned were carried out with a few lots of aggregate it would be extremely necessary to put in force paragraph 23 of the Report, by which the cost of testing should be provided for in the contract. He was afraid all these tests if so carried out would form a very large proportion of the cost of the work.

He was particularly struck with the statement on page 4 of the report, where it was suggested that the voids should be ascertained of (1) the whole and (2) of each separate grading. That was really carrying it too far. The same thing appeared on the next paragraph : (c) The proportion of each grading to the whole, (e) the specific gravity of the coarse material and sand, (b) the exact dimensions of specimen cubes, (c) the weight per cubic foot of all specimens immediately before testing. Now, that all seemed to him to be more or less needless expense. The same remarks applied more or less to the dates at which the cubes should be tested. The minimum tests suggested were from seven days to twelve months. Well now, surely, if one waited twelve months before getting a final test the work had all been done, good or bad, and it could not be altered. It seemed to him that anything beyond a twenty-eight days test for control purposes was unnecessary. It might be useful for research and for data afterwards, but for control purposes surely anything after twenty-eight days was not much use.

Mr. A. O. Trechman, F.C.S. — Personally he was very much against the testing of cement in a neat state. He had supervised the testing of many thousands of blocks, and thought it was only accumulating a huge mass of detail to record neat tests. Cement is purchased as a cementitious material, and it was its cementitious value that should be arrived at, and that was not arrived at by testing cement neat.

He was a believer in the crushing test, and thought it ought to be done more systematically and more regularly. In different works he had carried out a good many crushing tests and compared them with the tensile tests with cement in the course of manufacture, and the results were very interesting.

Mr. R. N. Sinclair. — In the first page of the report pit gravel was included in the coarse material. Did the Committee consider the advisability or otherwise of thoroughly examining this pit gravel, and if necessary washing it before it was used?

Of course, one knew the loam and clayey matters attached to pit gravel naturally when it is got out, and in the majority of cases it was not sufficiently good simply to take that ballast and put it into the concrete.

Coming to the question of sand, he thought that confining oneself to an aperture of 1-50th of an inch if they used sea sand they would lose the sand altogether. It seemed to him that 1-50th by 1-50th of an inch was certainly a large mesh to use for sea sand.

He would like to ask the Committee if they had absolutely decided that sea water was not to be used and that fresh water was the water to use? Contracts were made at the seaside for docks, and perhaps contractors would get a bucket overboard, or get down to a quay and get a bucket overboard if they were not watched. He would like to know if the Committee had come to a definite conclusion on that.

Had the Committee considered the question as to how long a time should elapse after the concrete was deposited in the moulds or on the shuttering before that shuttering was removed? Of course, it was an important thing for a contractor: if a contractor had a large area of floor and girders and beams to do it really was an important question to him, the question of this centering, and when a price was asked for it was the first thing he looked at. There was no mention here of that, but it was a question they were face to face with in actual practice.

Mr. Percival M. Fraser, A.R.I.B.A. — As a member of this Committee he would just like to try and answer one or two of the queries which had been put forward. The last speaker had rather mistaken the function of this Committee. It was not in any sense drawing up a specification of workmanship and materials. It was purely a question of testing. An engineer has certain materials made and unmade, and he had to put them to a test, and this report lays down the general rules that govern his testing of those materials.

The last speaker mentioned the period for striking centering. That was not in any way a matter for this Committee to report on.

He spoke, quite rightly, as to testing for cleanliness. That is an omission from this report. It was a very vital thing to get your sand and aggregate quite clean, and it should be emphasised in this report.

It is often said that fine stuff in sand is no detriment to the sand, but he thought the Committee were wise in fixing the minimum size of 1-50th in. by 1-50th in. All stuff retained at that to be rejected; probably a little more. In strict theory a little below that size might be efficacious, but the danger was that you may get a great deal below that size, which was distinctly detrimental.

With regard to determining the amount of water, the point was not laid down in hard and fast and dogmatic rules. Of course, the amount of water should be determined, but it should be determined by actual practice on one's work.

On page 4, section 21, sub-section (e) it said :—"All laboratory-made test cubes shall be made as far as possible on practical lines." But there was a value in a purely laboratory test. It showed the acme of strength which could be obtained by those materials if they were really mixed on scientific lines. He did not care for laboratory-made tests unless they were frankly made for that purpose.

The seven days' test was not usual for concrete. Most people seemed to think that fourteen days was the first valuable period at which one could test. He thought a seven days' test was absolutely essential. Reinforced concrete in these days was generally rushed through pretty quickly; we want to find out equally quickly what kind of material we are dealing with.

A gentleman suggested the elimination of neat tests of cement, but that was going a trifle far. We have learned the bearing of neat cement at 750 lb. to the in.; we have learned by indirect experiment the result likely to be got from a neat cement which stands a tensile trength, and until we had educated ourselves up a little more he did not think quite we ought to drop it.

Mr. J. B. Travers Solly, Assoc.M Inst.C.E. — With reference to the size of sand, he understood that this report was not supposed to deal with materials used in concrete, but only with the question of testing the materials. But still the Committee had made the remark that it was important that all sand not retained on an aperture of 1-50th in. by 1-50th in. should be rejected. Well, concrete work was not all done in England, and in some places it was very difficult to get the carefully graded sand that was advised by the Committee. And with reference to that he would like to mention that in some cases it might be better to use a sand that had a comparatively large proportion of fine dust than to use a drift sand with round grading. He had the opportunity of testing a sand under these conditions. The mortar was 2 to 1, 1 cement, 2 sand. He had drift sand, which was very rounded, and crushed sand, which had a very large amount of flour. The briquettes were tested at seven days and fourteen days, and he had three briquettes each test. With ordinary mixing, starting with drift sand, that was a rounded-edged sand, and cement, from that to the crushed sand containing flour, ordinary mixing, at seven days he got an increase of 29 per cent. in strength, tensile, of course. With the same crushed sand, using more water and mixing it very thoroughly, he got an increase of 69 per cent. At fourteen days he got similar increases of 20 per cent. and 34 per cent. and he attributed it entirely to the fact that the crushed sand was sharp-edged sand, whereas the drift sand had no sharp edge. He thought in every case the quality of the sand could be very easily judged under a magnifying glass, and that ought to be taken into consideration in connection with the question of passing a sand that has fine grading.

With reference to the testing of briquettes, or rather the condition in which the briquettes should be kept, it was specified in the report that all specimens should be kept in air after mixing, and slightly damp for the first seven days. He did not know that that was the usual system. Up to the last time he was doing any testing he covered the briquettes with a damp cloth for twenty-four hours, then took them out of the moulds and placed them in water, and they were kept in water for the six days, making seven days in all, and then tested.

Mr. Arthur Wm. Bangard, L.R.I.B.A.—He found in the report that the tensile strength of the steel required was given, but no strength was mentioned as to the crushing or the tensile strength of the concrete. Had the Committee left that out, or did they not intend giving it? If they gave it for the steel why not for the concrete?

In the second place, with regard to the cubes, the work would be done by the workmen, and it seemed to him that the cubes should be made on the spot, and the tests should be made on those cubes and not on a laboratory test at all. It was possible perhaps for a test on the laboratory-made blocks to be far higher than on those made by the navvy.

Mr. Yeatman.—He had omitted to ask whether Clause 18 and 18A, providing for a large percentage of elongation or contraction of area at fracture, did not exactly bear out the use of a higher tensile steel which would stand more than 60,000 lb. per sq. in. would not elongate so much as the figures. He understood that the tests on the twisted square bars, even though they were made of steel of about that quality, show that the twisted bars had a considerably higher ultimate strength and a very much higher yield of elasticity. They would stand about 60,000 lb. before showing much elongation, but he did not think they would expand 25 per cent. before fracture, but they make a very good material for reinforcement, and could be safely subjected, he thought, to a higher strain than the ordinary medium steel bars.

Mr. Roberts.—Might he ask whether there were any tests in regard to the shrinkage of concrete? He had never been able to get a definite result from it. He was sure that it would be absolutely impossible to carry out these tests and get anything like the cost they were supposed to have to do it at. He was speaking now as a contractor and knowing the competition.

Then take, for instance, the screening of the coarse material. In Thames ballast the amount of material that he had to screen had to pass a $\frac{3}{4}$-in. mesh. He only got about 25 per cent. of the bulk through the screen. But because of the rig of the screen he did not get anything like the percentage of aggregate or stone he would get through if he had the proper facilities for screening.

Mr. Trechman.—The question of the quality and the size of the sand had been raised. First of all, the cleanness of the sand was very, very important, and some test ought to be applicable or defined by which the suitability of the sand, its freedom from loam, should be arrived at; and, secondly, as to the size of the sand. He thought in preparing a report of this kind one was rather apt to assume that every kind of good material was put on wherever the job was. But supposing now that sea sand had to be used for a job, it was very likely, as one of the speakers suggested, that all that sea-sand would pass through the 1-50th by 1-50th in. sieve. Now, if that was to be used would it not be as well to specify the increased quantity of cement to be used in that case? The number of points would be very greatly increased, the surface area would be very greatly increased, therefore, the effect of the cement very much reduced.

The Chairman (Professor Henry Adams).—Before calling upon Mr. Alban Scott to reply he would like to say that they had had a very useful discussion, and he hoped the Committee would very fully consider the various points that had been raised.

In order that the work might be reliable and according to calculations, of course, it was necessary to make careful and uniform tests of the materials employed, but it appeared to him that some of the tests proposed were more in the nature of tests for the purpose of research than for the stability of the work. Remarks had been made about pit gravel. Well, pit gravel was once either river gravel or sea gravel, and the difference in nearly all cases was that there was a certain amount of iron-oxide attached to it, and often loam, and when pit gravel was used it was specially necessary to see that it was clean.

With regard to the question of grading the sand, he did not think that was so important as grading the larger material, but it enabled them to use some larger particles of sand than perhaps had formerly been the custom.

Mr. Alban Scott.—That is right.

The Chairman.—I suppose all the salt was washed out of it by that time?

With regard to the contraction of area, or the elongation, the tendency appeared to be at the present time to prefer the test for contraction to that for elongation, possibly because there used to be at least three standards of length, 6¼ inches, that is 100 16ths of an inch, 8 inches and 10 inches, and then later some shorter lengths still were put in, so that the percentage of elongation would vary with each of those lengths, although it was the same identical material.

The working in and "punning" and "tamping" and "ramming" were varieties of very useful processes, but he thought possibly the Committee might be able to improve upon the wording they had given to it there.

The Committee recommend that a provisional sum should be included in all contracts for the cost of testing. That was a counsel of perfection, but they gave no idea of what that cost will be.

With regard to testing the finished structures, very little had been said about that in the discussion. He thought that was open to grave risks. Suppose the structure was in such a condition as to need the test, then the greatest risk was caused, because you were straining parts beyond their elastic limits, we will say, or creating incipient cracks which did not fail at the time. Perhaps the point was clearer in the case of machinery. It came to him very forcibly a year or two ago, when the Home Office were formulating their regulations for docks and riverside warehouses as to the care of chains and machinery for lifting. They argued that the machinery should be tested with the full load periodically, and that that would be satisfactory. He advised them that, in his opinion, that would be most unsatisfactory, because it would lead to this : A machine might be tested (we will say it was doing nothing but being tested) time after time with the full load until ultimately it would give way with the full load. But the time before that, when it was tested, it took the full load and did not fail; therefore it is no safeguard, and particularly in the case of crane chains. He had had a very large quantity of these through his hands, and the responsibility of those chains and the men working under them, and he was, from experience, always against the repeated testing of a chain by mechanical tests. His practice was to have the chains periodically taken off, fired—that was, annealed—and examined link by link, but not put under a mechanical test; and he believed that the examination in that way was a much greater safeguard than a mechanical test. And so with regard to testing the finished building. It was very much better to see that you had proper supervision, proper designs, and proper checking of those designs before the work was put in hand than to rely upon any tests that you might make upon the finished work afterwards.

REPLY TO THE DISCUSSION.

Mr. Alban Scott.—He could not reply quite in the sense of a usual reply to a paper, because he was not in any way responsible for this Report, except as one unit of the Committee. One thing struck him as very peculiar. Only one gentleman that evening had criticised the question of testing steel. From their silence he concluded they thought it desirable to test steel, therefore he could not understand why they objected to testing concrete, as many of them did—on the ground of expense, generally. There was more human element employed in the making of concrete than in the making of steel, and, therefore, there was more chance of defects. The specification put here, or suggested specification, in the way of tensile steel was now accepted, practically, by every rolling mill, provided you give them good notice that they could get proper billets to-morrow. Every contractor would accept this specification for his reinforced concrete, for this reason—that on March 8th he gave a paper on this subject at the Society of Arts, and one point there he tried to make particularly clear was, that at the present moment the different official rules are taking 1,800 lb. as the ultimate strength of concrete and are taking 600 lb. as a working load. If you dragged up to 600 lb. in compression for your concrete you must have the very best concrete, and if you are not to have the very best concrete then you had better leave it alone altogether.

Professor Adams said it was only a few years ago when the question of a ¾-in. aggregate was beyond realisation, but now one could get aggregate of sand graded into any size and any proportions for the cost of about sixpence a yard extra.

Of course, the idea of sorting out Thames ballast and only getting 25 per cent. was a thing of the past, years and years old. If they would not have their concrete tested and were to rely on haphazard methods, all he could say was they could very shortly anticipate terrible failures in this reinforced concrete work. They had had a few, but they were going to get more, unless the R.I.B.A. and the London County Council alter the limit of 1,800 lb. and reduce it to at least 1,500 lb. You could not, on any job, no matter how carefully it was done, get a higher average than 1,500 lb. in concrete with the very best grading, with the very best cement, and the very best workmanship. Therefore, as long as they kept that high figure of 1,800 lb. and allowed one to work up to 600 lb. actual working stress, every care must be given to the work.

On a job which he was doing now, reinforced concrete work of about £10,000, they are spending £150 on testing, and that covered in principle every kind mentioned in this Report. He did not consider that £150 as the cost of testing on a £10,000 job was anything at all outrageous, or a price which anybody would grumble at. The contractors were now giving their professional people every help they could to get the work correct, because they realised that there was danger. One trouble seemed to be the extent of competition in prices; but when one came to consider the question of safety of the building it did not count.

On the question of cleanliness of gravel, we had had some most extraordinary results on some tested recently. Up to April 8th we only had one or two tested, and about 7 per cent. of loam was the average. Since that date we have had a good many tested, and found the loam in pit gravel varied from 5 to 29 per cent. of its bulk in weight, and the 29 per cent. stuff really looked comparatively clean. As compared with quite clean material, one would never realise the enormous amount of loam there was. That material was submitted for use in an actual structure.

The Chairman expressed the thanks of the meeting to the Tests Standing Committee for this Report, and particularly to Mr. Kirkaldy (the Chairman) and Mr. Alban Scott (the Honorary Secretary), because upon those two officials the work of the Committee chiefly fell.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE WHARF AT CORK HARBOUR.

The new Marina Wharf, which has recently been completed at Cork for the Harbour Commissioners, is another typical example of the use of reinforced concrete for Harbour works. In this case instead of applying directly for tenders from a certain number of contractors, licensees of the various systems, or from the specialists themselves, the actual scheme for the work was placed in competition between a few selected firms of specialist designers on a general design prepared by Mr. James Price, B.A.I., Chief Engineer of the Cork Harbour Commissioners.

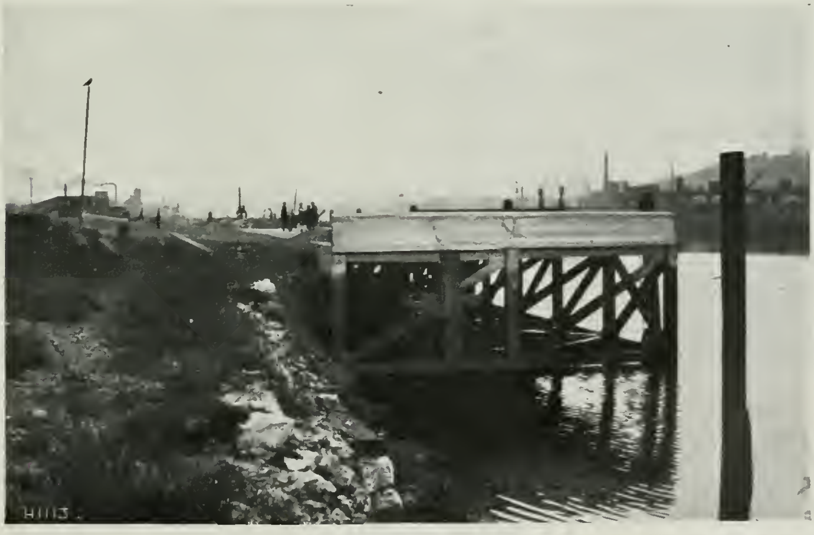
Each firm was required to send in a scheme based upon the rules of the Royal Institute of British Architects for reinforced concrete, and in accordance with the specification of the engineer. An estimate of the cost of the work was also required, together with a set of quantities and a description of the competing system.

After examining the various schemes, Messrs. Edmond Coignet, Ltd., were selected and appointed for the preparation of the necessary working drawings and particulars to enable the work to be carried out by the Commissioners with their own staff and under the supervision of their chief engineer.

This wharf, which has a total length of 500 ft., is situated at the eastern end of Cork City Quays. The wharf, which is of very simple structure, is composed of equal bays of 15 ft. 1½ in. in width, each bay being supported by principal and secondary beams. There are altogether thirty-three similar bays, and the total width of the wharf is 35 ft.

The deck was calculated for a superload of 3 cwt. per sq. ft., with a test load of 4½ cwt. per sq. ft.

Owing to the close spacing of the secondary beams, which are only about 5-ft.



Side View of Wharf.
Reinforced Concrete Wharf at Cork Harbour.

centres, the thickness of the slab required is only 4 in., but a greater thickness was constructed to allow for wear. The front piles are 40 ft. in length; the piles in the second row are 31 ft. in length, and those in the back row are only 15 ft. in length, longer piles being used in the back and centre rows where required. There are thirty-four similar rows of piles, so that altogether there are 102 piles.

Work in Course of Construction.

View of Finished Wharf.
Reinforced Concrete Wharf at Cork Harbour.

A retaining wall in reinforced concrete, with 30-ft. tie backs at each pile, was provided at the back of the work to retain the earth.

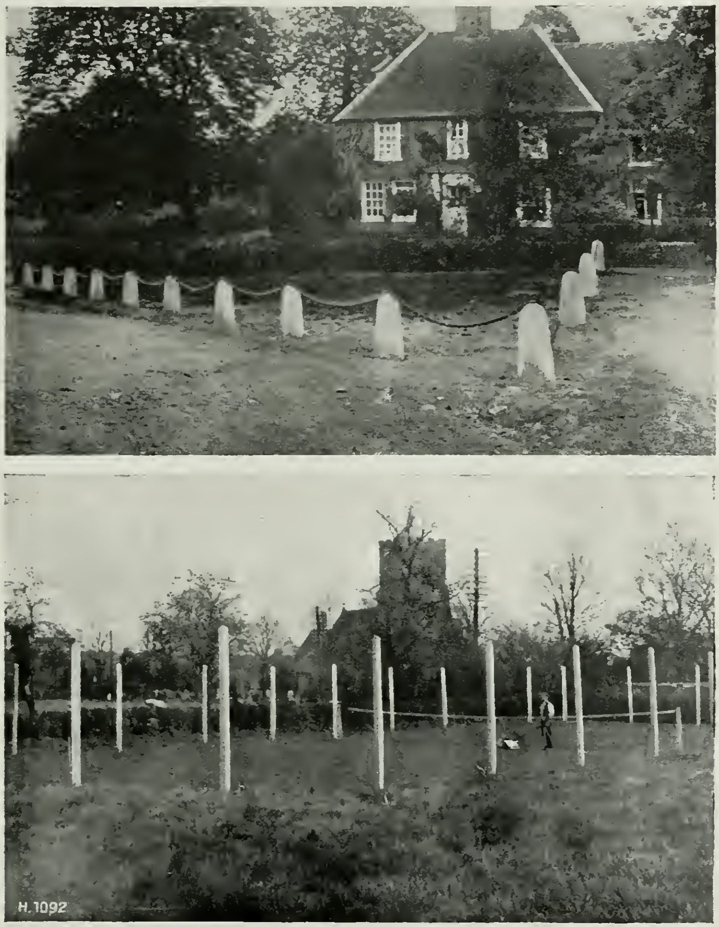
The wharf is strengthened by means of horizontal and vertical braces, as shown in the illustrations.

The piles were all made of circular section, having a certain number of vertical bars bound together by means of hoops, in accordance with the Coignet system. The driving operation was carried out by means of a steam monkey weighing 35 cwt., the driving apparatus being fixed on a barge.

The front of the wharf is protected by means of wooden fenders attached to the reinforced concrete work at deck-level, but kept 6 in. clear of the concrete at low-water level.

REINFORCED CONCRETE POSTS AND TENNIS POLES.

THE illustrations on this page are good examples of some of the uses to which reinforced concrete can be put for ornamental purposes, where strength and durability are also required. The upper view shows some reinforced concrete posts, the lower one illustrating reinforced concrete poles for a tennis court. The posts in both cases were manufactured by the Reinforced Concrete Fence Posts Co., Ltd., of West Mill, Buntingford, Herts.

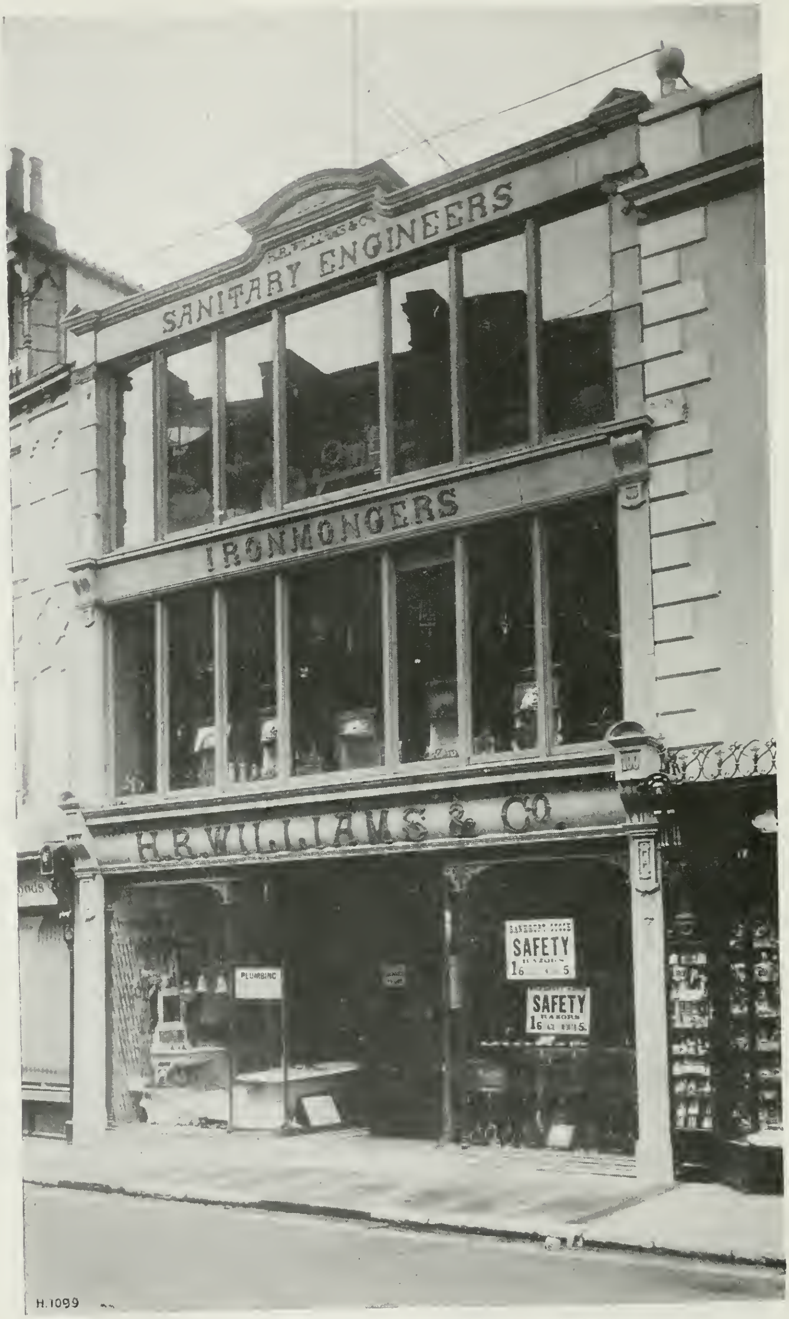


Reinforced Concrete Posts and Tennis Poles.

REINFORCED CONCRETE PREMISES IN HIGH STREET, BARNSTAPLE.

THIS building was erected as showrooms for Messrs. H. R. Williams and Co., ironmongers and sanitary engineers of Barnstaple, and was necessitated by the nearly total collapse of their then existing premises.

The building, which is a three-storey one, is constructed of reinforced concrete, the whole of the floor weights being transmitted to pillars on either side of the site, and these, owing to the weak state of the adjoining properties, rest on concrete



View of Finished Building.
Reinforced Concrete Premises, High Street, Barnstaple.

foundation beams going across the building, instead of having the usual independent foundations or bases. The walls, except to front, are of concrete 4 in. in thickness reinforced with steel bars running into and bonding with the pillars.

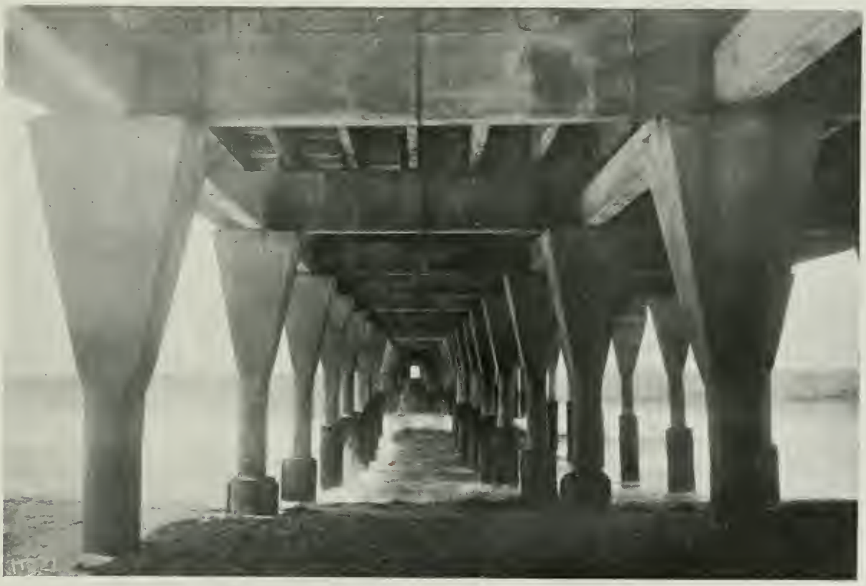
The pillars are 14 in. by 14 in. on ground floor and 12 in. by 12 in. above, and the beams are 18 in. wide and 16 in. deep, with secondary beams 13 in. by 8 in. for each floor and roof, which are also entirely of concrete and 4 in. in thickness. The concrete floors are covered with boarding and the roof with an asphalted felt. There is a lift serving the three floors, the pulley of which is supported by one of the secondary beams.

The building has a frontage to the main street of nearly 23 ft., tapering slightly towards the back. It forms an imposing and attractive front, being faced with faience ware of a cream colour fastened directly to the concrete and as a finish to the latter.

The whole of the building works were carried out by Messrs. J. Cater and Son, of Barnstaple. The designs were prepared and the building supervised by Mr. Spencer Edwards, P.A.S.I., M.C.I., architect, of Barnstaple.

REINFORCED CONCRETE PIER AT SANTA MONICA, CALIFORNIA.

While reinforced concrete has been largely used for the construction of piers, this is the first time that a reinforced concrete wharf has been built into the Pacific Ocean.



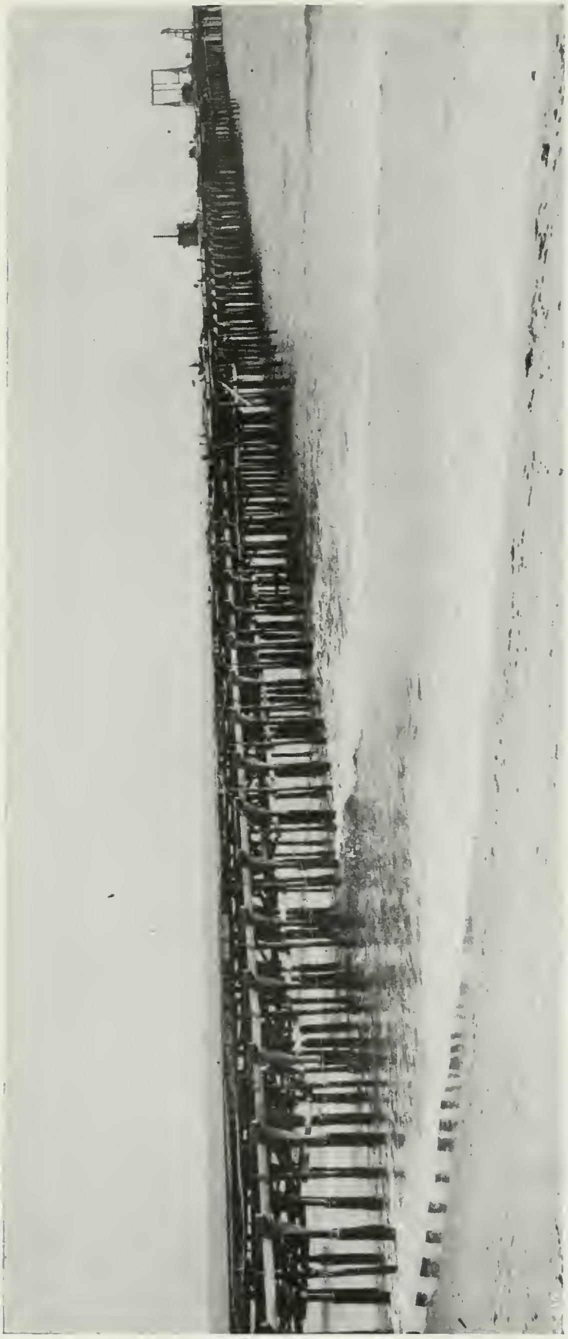
Under Side of Pier.
Reinforced Concrete Pier at Santa Monica, California.

The greatest confidence is expressed in its ability to stand the worst storms of the bay, and up to the present it has made good all promises.

The pier extends into the ocean for a distance of 1,600 ft. and has a width of 30 ft. No wood has been used in its construction, the piles, floor, struts, and girders all being of reinforced concrete. The end of the pier forms a large platform, ornamented with concrete, and there are two similar platforms in the middle of the bridge.

At first the pier was designed as a pleasure promenade, but later on it was decided to use it as a landing stage for small pleasure steamers. It has cost over $115,000 (£23,000).

The style of the construction being new, the contractors found themselves almost daily facing engineering difficulties of a trying nature. At first great boulders were encountered in the bed of the ocean when it was sought to sink the petrified trees into the sand. These had to be removed, and this was accomplished by the jetting of a stream of water about the great stones. In this way the obstructions were rolled out of the way. At another time the stones dropped into strata of quicksand. It seemed as if this yielding material was without bottom, but in every case the piles came to solid foundation before the required depth had been reached.



Pier in Course of Construction.

Reinforced Concrete Pier at Santa Monica, California.

In only one instance did the builders find it impossible to roll the boulders out of the way and to sink the pile to the depth desired. This one impediment could not be removed, and there was nothing to do but to permit the end of the pile to rest upon the obstruction. But before this was done ten barrels of liquid cement were forced by hydraulic pressure into the sand at the foot of the pile and the nest of boulders.

Hundreds of reinforced concrete piles were used in the construction work. The piles range all the way from 40 ft. to 70 ft. in length. In diameter they vary from 14 in. to 22 in., the first 500 being 14 in., the second section 18 in., and the third 22 in. in diameter; there are 280 piles in the pier.

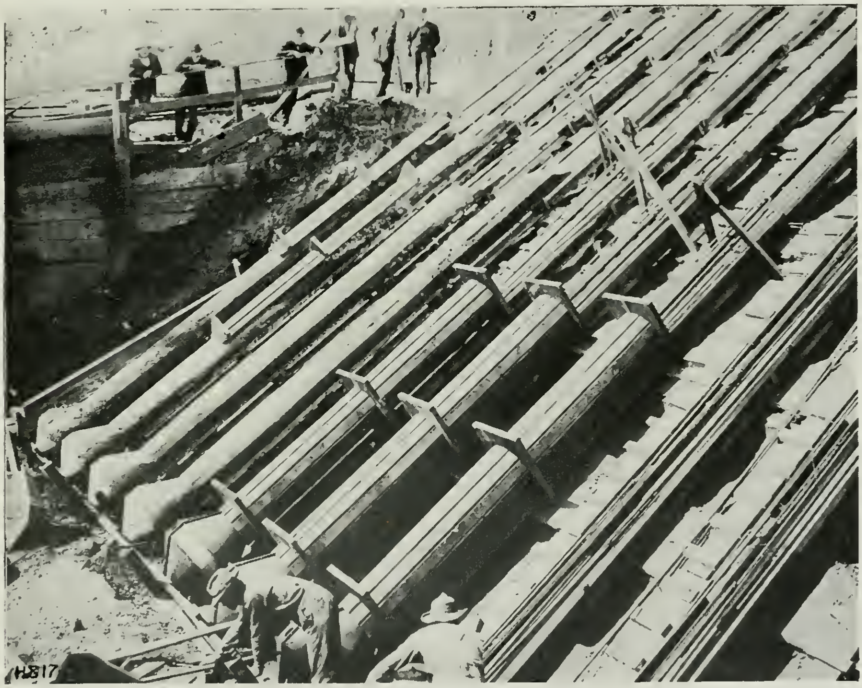
The piles were cast near the point where the pier was being built, and after being cured were conveyed on a temporary tramway and lowered into the sea. They were sunk into the sand from 16 ft. to 20 ft. apart. This was done by means of a water jet. Each trestle bent of the pier was made up of three piles spaced 13½ ft. apart. Each of the round piles was reinforced with from six to eight 1-in. steel rods, arranged symmetrically 2 in. from the outside of the pile and tied together every 3 ft. of length with wire to a 2-in. pitch.

There is a bulb point cast at the lower end of each pile, and into this the lower ends of the steel reinforcing rods are splayed. The upper ends of the rods extend through the bracketed heads of the piles, and were carried above the head of the pile to mesh into the reinforcement of the concrete girder and strut.

Through the end of each pile was run a 2-in. jet pipe with a hose coupling connection in jetting. After the pile had been placed in permanent position this pipe was filled with concrete. After this, each pile was enclosed in a jacket of No. 12 steel. These jackets were 9 ft. long and extended from 3 ft. below the line of mean tide to 6 ft. above.

The longitudinal struts and girders are of reinforced concrete, bolted and meshed into the bracketed heads of the piles by means of steel rods. The floor of the pier is of asphalt. At the point where this pier was being constructed the slope of the water is very gradual, sloping seaward at the rate of about 2 ft. to every 100 ft.

The photographs from which our illustrations were made were placed at our disposal by Mr. Fr. Bock, engineer, of Charlottenburg, Berlin.



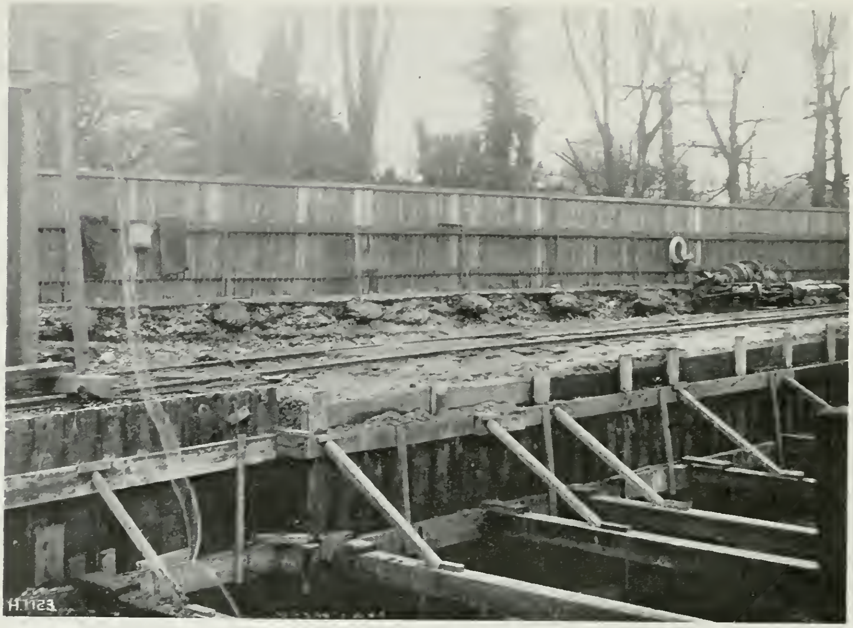
Forms for the Reinforced Concrete Piles.

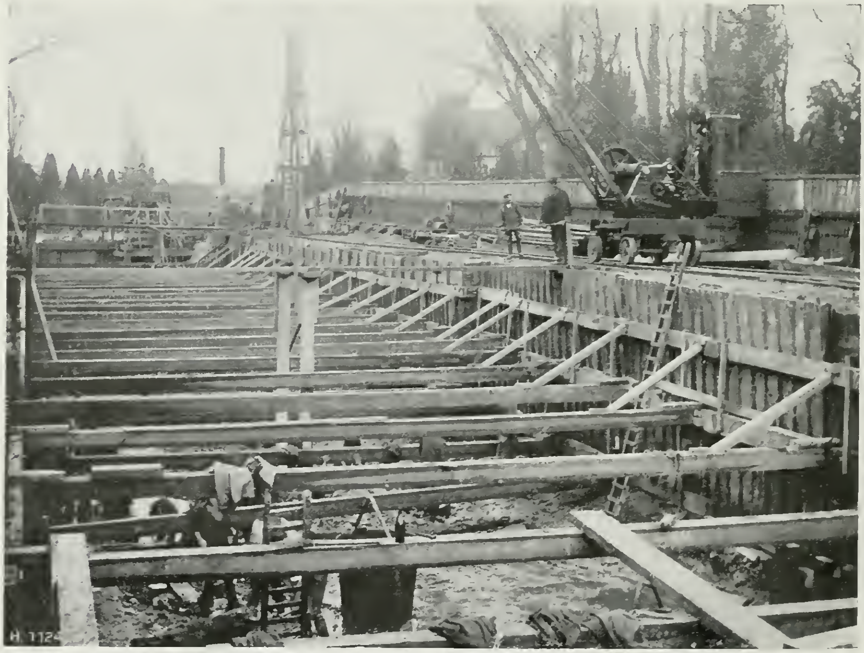
Reinforced Concrete Pier at Santa Monica, California.

STEEL SHEET PILING IN THE RECONSTRUCTION OF BOULTER'S LOCK.

To enable this work to be carried out expeditiously a quantity of the British Steel Piling Company's "Universal" joist steel sheet piling, 15 in. by 5 in. section, weighing 43 lb. per super ft. when interlocked, was driven along the entire length of the western side of the lock behind the part to be excavated, so as to shore up the ground to prevent subsidence around and to assist in keeping water out of the excavations. Like piling was also driven along the eastern side at the upper end, where the new lock was to extend beyond the limit of the old one.

The existing masonry wall on the east side being too close to other property to permit of its demolition without risk of interference, it was left in, and the eastern line of steel piling was driven in continuation of the wall northward to the limit required for the new lock construction.





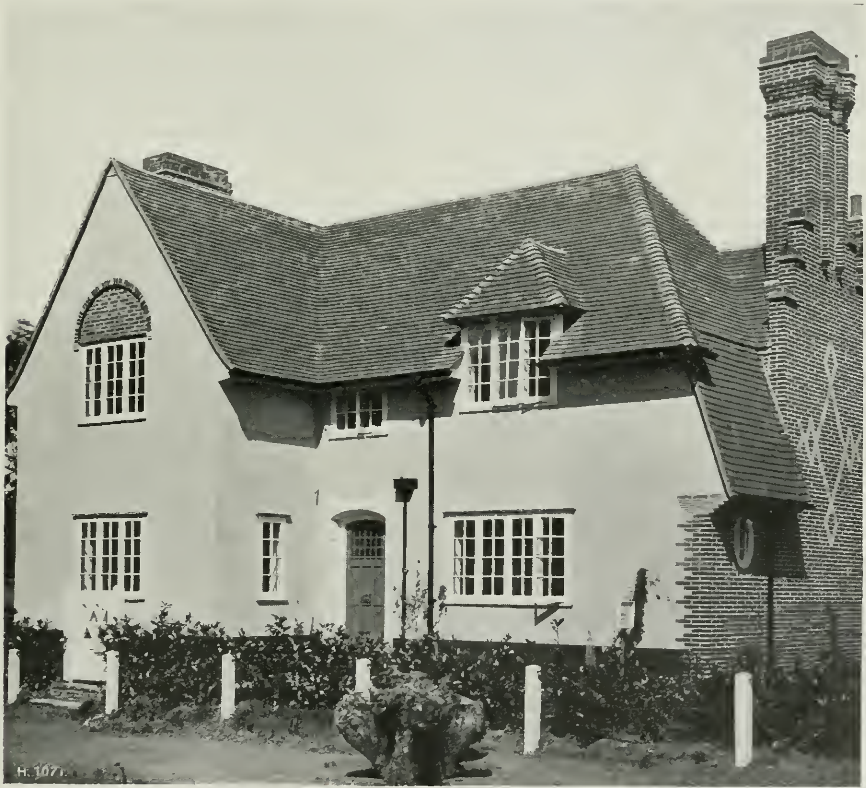
View Showing Steel Sheet Piling in Use.
The Reconstruction of Boulter's Lock.

The illustrations on page 632 show the steel piling in use. The piles were in lengths of 20 ft. and 25 ft., and were driven through Thames ballast and running sand to their full extent. The piles were perfectly watertight. To drive them a patent steam pile driving plant with a steam hammer weighing 20 cwt. (all of the British Steel Piling Company's pattern) was used.

The piles took from 4½ to 40 minutes each to drive, or on the average 20 minutes each, which latter time was also the average required to withdraw each pile after the completion of the work. This undoubtedly represents a very considerable saving in time and expense as compared with timber sheet piling, and, in addition, much less disturbance of the ground was caused.

CONCRETE BLOCK HOUSES ON THE GIDEA PARK ESTATE.

In our June number we gave an illustration of one type of concrete block house on this estate, and we now show a view of another of these houses of a slightly different pattern. All these houses are being built by Messrs. Henry Lovatt, Ltd., of London and Wolverhampton, with "Winget" concrete slabs. With the exception of the chimneys, the whole house is built of "Winget" blocks and slabs.



Concrete Block House on the Gidea Park Estate.

NEW BOOKS

AT HOME AND ABROAD.

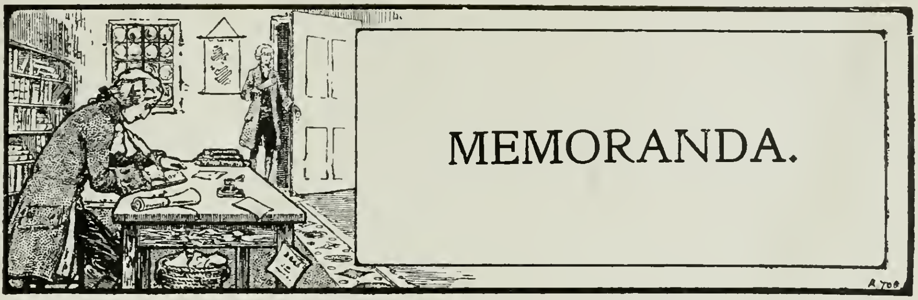
A short summary of some of the leading books which have appeared during the last few months.

"Theory of Structures." By Arthur Morley, *M.Sc., M.I.Mech.E., Professor of Mechanical Engineering in University College, Nottingham*

Published by Longmans, Green & Co., 39 Paternoster Row, London. Price 7/6 net. 8vo., 574 pp.+xii.

Contents.—Stress and Strain—Working Stresses—Statics—Bending Moments and Shearing Forces—Stresses in Beams—Moving Loads—Deflection of Beams—Elasticity of Beams—Direct and Bending Stresses—Framed Structures—Stresses in Frames—Moving Load Stresses in Frames—Selected Typical Framed Structures—Deflection and Indeterminate Frames—Some Indeterminate Combinations—Frame Members and Structural Connections—Plate Girders and Bridges—Suspension Bridges and Metal Arches—Earth Pressure, Foundations, Masonry Structures—Dimensions and Properties of British Standard Sections — Mathematical Tables — Answers to Examples—Index.

Professor Morley's companion volume, "Strength of Materials," has now been before the public some few years, and we have a great admiration for it. Though it relates to the laws governing the resistance of materials to stresses appearing in either machines or structures, it is chiefly concerned with the latter class of constructions, and we believed it to be the best book published so far on the theory of structures, for, though it was professedly limited in scope, it covered a large field in a fashion so greatly superior to other books that its limitations were almost overlooked, though we always regretted that Professor Morley should have stopped where he did and should not have continued his masterly method of treatment further in the direction of structural engineering. The present volume does that, however, and we give it a hearty welcome. A good deal of information is repeated from the other volume, for, as the author says, "to make this book serviceable to the reader who is concerned with structures only and not with machines, sufficient of the theory of stresses and strains in single pieces has been included to make it complete in itself." It would be a pity, however, if the earnest student were led by this to purchase only this volume, for there are many important questions that arise in structural engineering which are dealt with in the first volume alone, and the two are so much companion volumes that we almost regret that the author did not make them one, and so avoid what little duplication there is. The two books are the best we know of as class-books for engineering colleges, and at the same time are invaluable assistants to practising structural engineers, and this volume particularly contains examples from actual practice. Professor Morley thinks reinforced concrete deserves a separate volume, so he does not deal with it in this book. We hope, however, that he will deal with it before long, because he ought to be able to clear up much that is in doubt and at present treated empirically, if he would only concentrate his mind on it and consider it *de novo;* and we feel sure that when he does he will find it act in turn upon the theory of ordinary structures to a surprising extent and lead him to go into many tricky points that he at present omits to deal with. There are few enough theorists of first order at work on the theory of structures for us to wish Professor Morley would continue to give his energies to the subject and not be drawn aside into the theory of machines.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

Methods of Depositing Concrete Under Water.—This was one of the subjects reported upon by the Committee on Masonry at the annual meeting of the American Railway Engineering Association. The replies to a circular letter of inquiry covered nine methods, as follows: 1, bottom-dump bucket; 2, tremie (or closed chute); 3, open chutes; 4, sacks; 5, sacks withdrawn; 6, paper bags; 7, open depositing; 8, pneumatic grouting of broken stone; 9, concrete blocks. From an examination of the replies the committee has drawn the following conclusions:—

1. Concrete may be deposited successfully under water if so handled as to prevent the washing of the cement from the mixture.
2. Cofferdams should be sufficiently tight to prevent current through the pit, and the water in the pit should be quiet.
3. The concrete should be deposited in place either by means of a drop-bottom bucket or a tremie, and should not be allowed to fall through the water.
4. Where a bucket is used, it should be carefully lowered to the bottom and raised to the surface, so as to cause as little disturbance as possible.
5. Where a tremie is used, it should be kept filled with concrete up to the top of the water level; the discharge end should be kept buried in the freshly deposited mass to prevent emptying, and raised a few inches at a time as the filling progresses.
6. The surface of the concrete must be kept as nearly level as possible, to avoid the formation of pockets, which will retain laitance and sediment.
7. Where concrete is not deposited continuously all sediment should be removed from the surface of the concrete, by pumping or otherwise, before depositing fresh concrete.
8. The concrete should be a 1 : 2 : 4 mixture and of a "quaking" consistency.
9. Freshly deposited concrete should not be disturbed.
10. In open water, or where flow of water through the pit cannot be prevented, concrete should be deposited in cloth sacks.

Protection of Concrete Against Freezing.—Protection of concrete against freezing was accomplished in the case of a large reservoir at Virginia (Minn.) by covering the exposed portion with a layer of clay, then a layer of cinders, and above that a second layer of clay—making a total thickness of 18 in. The reservoir, which was built for the Virginia Electric Power and Water Company, was 45 ft. in diameter and 21 ft. high, and was partly above and partly below the ground. When the covering was removed in the spring, the tank showed no sign of injury by frost, in spite of the severe winter; and the concrete appeared to be entirely water-tight.

A Water-tight Concrete Tank.—A water-tight concrete tank, with walls 6 in. thick, was obtained by using a rich concrete mixture with good quality, carefully graded materials, in some recent construction work on the El Paso and South-Western Railway. The tank is 30 ft. in diameter and 12 ft. high. The concrete was mixed 1 : 2 : 4, or a little richer in cement; and the gravel was required to pass a ½-in. screen. No waterproofing was used. It is stated that the tank is entirely water-tight except at joints between the courses of concrete. Another reservoir with a concrete lining 4 in. to 6 in. thick, built more than three years ago, and waterproofed with soap and alum has proved water-tight during the entire period. The sand and gravel in this work are stated to be of fair grade only, the gravel reaching a maximum size of 1½ in.

Surface Treatment of Concrete Walls.—Surface-facing coats of rich concrete for concrete walls are not as satisfactory as walls having uniform mixture throughout, according to the experience of Mr. Lincoln Bush, consulting engineer, New York, related in a memorandum to the Committee on Masonry of the American Railway Engineering Association. His observations on walls built for the Lackawanna Railroad during the period from 1900 to 1909 have shown that walls built with a uniform 1 : 3 : 5 mixture between 1903 and 1909 have given better results than those built between 1900 and 1903, when the practice of that road was to use a 2-in. surface finish of 1 : 2 : 2 mixture against a body of 1 : 3 : 5 concrete. Defects and shelling-off of the face may not take place for two or three years, and may then continue. Where a special surface finish is necessary for ornamentation, he believes it best to obtain it by bush-hammering, treatment with acid, or rubbing and washing while the concrete is green, using crushed stone of proper size to give the desired surface effect.

Scientific Cost Records in Construction Work.—Ignorance of real costs is a danger whose seriousness every contractor will appreciate. What is less clearly appreciated is the extent to which real costs are not known, and the degree to which construction estimates are based on gravely unreliable data. Discussing this matter, Mr. Sanford E. Thompson, consulting engineer, who was associated with Mr. Frederick W. Taylor in writing " Concrete Costs " (reviewed in our July number), says :—

Besides being of use as a preliminary step toward the introduction of scientific management, cost keeping—that is, cost determination of work in progress—is of value to the engineer for making up estimates and checking the work of the builder, and to the builder in bidding on subsequent contracts and keeping track of the cost of the work as it progresses from day to day. In construction work based on the principle of cost plus a fixed sum, and other similar systems, the accurate recording of detail costs on different parts of the work is absolutely essential for submitting the accounts to the owners.

To accomplish any of these aims, the cost records must be accurate enough to serve—

1. As records for estimating the cost of subsequent jobs.
2. For immediate use :
   (*a*) To determine whether the builder is making or losing money;
   (*b*) To fix any point of loss or of too small profit;
   (*c*) As an incentive to the foreman and workmen.

As generally practised, cost keeping is so approximate and inaccurate as to be of comparatively little value for any of these purposes.

The point just made may be illustrated simply by a comparison of the methods now usually employed in estimating materials and labour. In estimating materials the engineer or contractor notes every item, usually taking the schedule from the plans, and, by adding a percentage for contingencies, reaches a total which will check fairly well with the actual subsequent cost. Before he starts to do any work he must order the required amount of each material separately, and the cost of each item is carefully looked into to see that the lowest figures are obtained consistent with the quality of the work required.

With labour, on the other hand, the plan heretofore adopted has been largely a system of guess work. Frequently one hundred or more carefully tabulated material items are set down, while the estimate for labour is given in one lump sum, and yet the labour may amount to one-fourth or one-third the sum total of the materials. The variation in the actual cost of the labour from that given in the estimate almost always makes the difference between a profit and a loss to the contractor. These " guesses " at labour costs are commonly excused because it is claimed that the work done by different workmen varies to a great extent, or that it is impossible to provide for unforeseen contingencies. This, however, is merely dodging the whole responsibility. The real reason for such approximations is that the contractor usually does not know the time and cost of doing each kind of work with any degree of accuracy; and the fact is only just coming to be recognised that it is possible to determine in advance how fast each element of the work should be done nearly as accurately as the cost of supplies and materials are now determined, and that once having the fundamental data, it is possible to estimate labour nearly as accurately as material.

Concrete Radiators.—A radiator for steam or hot water constructed of concrete has recently been invented in Germany. According to *Cement Age*, these radiators are manufactured by pouring a mixture of cement and sand into special gypsum moulds or iron casings. The thickness of the walls is about $\frac{3}{8}$ in. The specific gravity of the concrete material is low and in consequence 10 sq. ft. of radiator space weigh only 20 kg., which is hardly more than half the weight of iron radiators. The advantages from a sanitary point of view are to prevent the formation of dry air in a heated room. The concrete radiators are porous, allowing moisture to go through. They heat more quickly and cool off more slowly than iron radiators. They do not rust and can therefore be connected to hot-water heating, and they cost much less than if made of iron. Numbers of concrete radiators in all possible forms and colours were shown at the recent building exhibition in Dresden.

Concerning the Difference in Concrete.—In a recent number of *Engineering News* the following remarks occurred :—

"There is concrete and concrete! That is a fact which should be clearly understood by every engineer whenever the subject of concrete is under discussion. We have been especially impressed recently with the contrast between the quality of the average concrete, such as one sees mixed and placed by the average small contractor and many large contractors as well; such concrete as one often sees which, after having stood a few years, shows more or less places where defective finish, disintegration and cracking are apparent—the contrast, we say, beween such average concrete and the concrete which is being made on some engineering works of large size, where such concrete as an engineer describes in his specifications is actually produced. Two notable examples of such ideal concrete work are the Panama Canal locks and the great dam, power-house and lock of the Mississippi Power Co., now under construction at Keokuk, Iowa. In making concrete on these works, good stone, good cement, and good sand are assembled, and a measured quantity of each is deposited in a large batch mixer and thoroughly mixed, with enough water to make a sloppy mass, in which air bubbles will inevitably rise to the top. The mixed concrete is then taken in large buckets and deposited inside substantial forms. The work goes on so rapidly that the form is filled to a considerable depth before setting begins.

"It is probable that one of the chief reasons for the remarkable excellence of concrete made in this way is the fact that the soft concrete is put under a heavy hydrostatic pressure before setting begins, and is thus made far more dense than was ever possible under the old practice when a dry mix for concrete and thorough ramming were universally specified by engineers.

"Of course, it is well understood that a perfection is possible in the concrete work on huge structures which is out of the question on any ordinary concrete job; but certainly the way average concrete is laid leaves large possibilities for improvement. It is very difficult to make the average small contractor or concrete workman take the pains to make high-class concrete, for he has found by experience that making it with any old materials any old way produces results which, so far as he can see, are good enough. Often it is not until the engineer can point to curbs battered to pieces, sidewalks cracked, and retaining walls weeping at the courses or bulging from weak forms that he can secure the enforcement of his specifications to the letter. There is a scarcity, without doubt, of concrete contractors who will execute work in the most thorough manner, and of concrete workmen who will give conscientious care to their tasks. But, given such contractors and such workmen, achievements are possible in concrete work which many engineers hesitate under present conditions to undertake."

Lectures on Reinforced Concrete.—On July 3rd and 17th Mr. W. Noble Twelvetrees delivered two lectures on reinforced concrete to the students of the Crystal Palace School of Practical Engineering. The first lecture dealt with the fundamental principles of reinforced concrete construction and the general theory of beam design. The second lecture was devoted to the principles underlying the design of columns, struts, arches and other forms of structural elements, and to various aspects of practical construction in reinforced concrete. This lecture was illustrated by specimens of various types of reinforcing steel, as well as by models and photographs of works executed in the United Kingdom.

The British Fire Prevention Committee's Summer Meeting.—This Committee held its two-day summer meeting on July 24th and 25th at its Regent's Park Testing Station, when some important high-temperature fire tests on a reinforced concrete floor; on electro glazed casements; and a reinforced concrete door were held. There was a large attendance of members and subscribers, as well as representatives from

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various Government Departments. The London County Council and several municipal and other authorities were represented by their principal officials, and a number of representatives of railway and insurance companies were also present.

The arrangement of the reinforced concrete floor test had awakened considerable interest, as it was the first occasion that a floor of this type, reinforced solely with a triangular mesh reinforcement (put forward in this country by the United States Steel Products Company), has been under official review, and we understand that the test was eminently satisfactory.

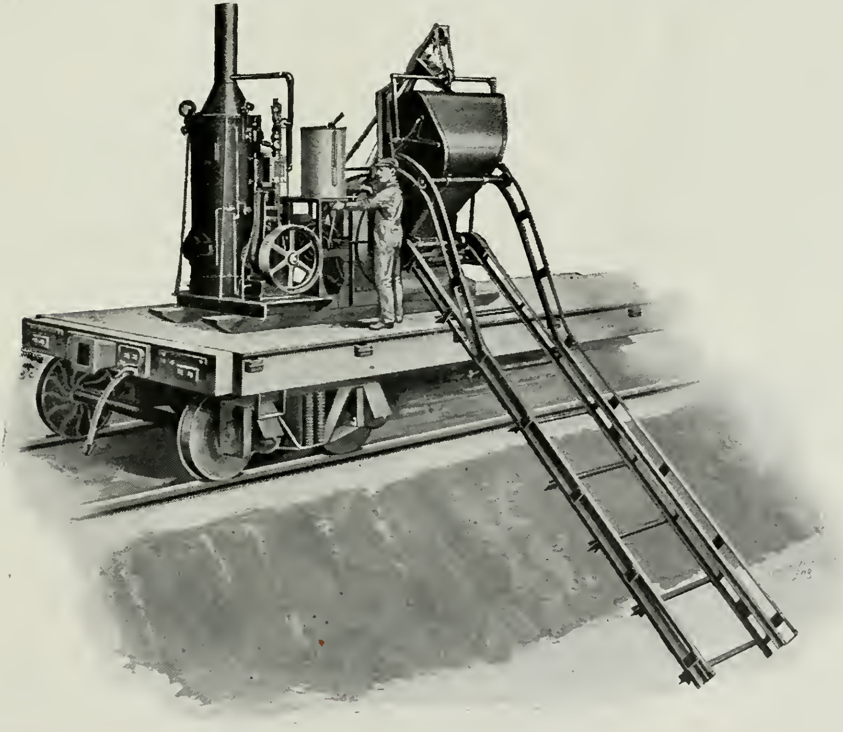
The double door constructed of reinforced concrete, which was tested on the second day, came from Belgium, where it had been constructed to the specifications of the Chairman of the local Government Fire Committee, and had been sent over for report.

In giving the above particulars we think we should point out the important place concrete is taking in these various questions of fire investigation. Not only was there a large reinforced concrete floor under test and a reinforced concrete door, but even in the glazing tests the lintels and mullions were of concrete, and the effect of the fire on these should be as interesting to study as the effect on the glazing actually under investigation.

As to the official results of the tests the usual illustrated reports will be issued in due course directly after the vacation.

TRADE NOTICES, CATALOGUES, ETC.

A Loader Extension for Concrete Mixers.—In construction work, it is frequently an advantage to allow a concrete mixer to remain on an embankment and have some means of lowering the loading bucket, so that it can receive its charge at some distance below grade. This is particularly the case in railroad construction work, where the mixer is on a flat car. To meet this demand a form of power loader extension has been designed by Bagshawe and Co., Ltd., Dunstable, Beds., England, manufacturers of Chain Belt concrete mixers, for application to mixers of any standard sizes. In the mixing plant shown in the illustration, which was recently sold to the George A. Fuller Construction Co., Ltd., the loader travels down about 15 ft. From experiments that have been made there seems to be no reason why these extensions cannot be made to operate successfully over a distance twice as great as that.



The main channels are pivoted at a point on the loading side of the mixer to permit the loader extension being adjusted to operate at any angle. The extension frame is so made that it can be detached at the pivot joint and replaced with a standard pivot bearing, so that if it is desired to use the mixer on the ground level all that is necessary is to detach the extension bearing and attach the regular pivot bearing, the same loader being used in both cases. In using the standard pivot bearing the wheels and rear axles on the power-loader bucket will be unnecessary and will have to be removed.

The First Cottbus Cement Goods and Machine Works have sent us their new prospectus which deals at length with their " Perfection " partition slab machine. A great advantage of this machine is that both hollow and solid partition slabs can be made on the same machine, in any thickness from 2 in. to 4 in., either with straight ends and sides or grooved and tongued as desired. Paving slabs and agricultural pipes can also be made on the " Perfection " machine.

These machines can be worked by one man, and their output is very large, thus ensuring a low cost of production.

The booklet, which is well illustrated, will be sent on application to the company's offices at 8, Manor Park, Lewisham, S.E.

ENQUIRIES.

S. B. (Runcorn) writes:—" Can you or any of your readers give me a simple (if only approximate) method of calculating the strength of a circular reinforced concrete slab supported all round? How would such a slab compare in strength with a rectangular slab of similar thickness supported one way across an equal span? "

—For distributed loading a circular slab supported all round may be taken for a maximum central bending moment of $\frac{w\,l^2}{16}$ in both directions where the bars are laid at right angles. For comparison, we may remind inquirer that a slab supported on two edges under distributed loading has a maximum bending moment of $\frac{w\,l^2}{8}$, so the circular slab is twice as strong. For circular slabs up to about 15 ft. in diameter there would be greater support of the central strip derived from the adjoining sides, and the bending moment might with safety be taken at $\frac{w\,l^2}{24}$ where l = diameter. The rods might be gradually reduced also as the sides were approached, *i.e.*, spaced wider apart.

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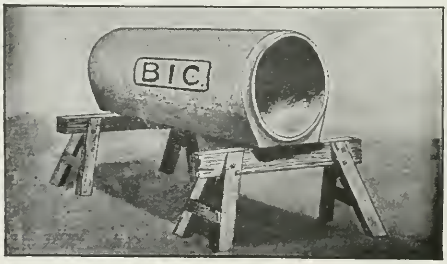


Porous Pipes
Concrete Beams
Hollow Floors

SPECIALITY:—INVENTORS' DESIGNS CARRIED OUT TO ORDER.

(*For description see page 674.*)

View of the Cavendish Bridge, Mauritius.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 9. LONDON, SEPTEMBER, 1912.

EDITORIAL NOTES.

THE INTERNATIONAL CONGRESS ON THE TESTING OF MATERIALS.

CONTEMPORANEOUSLY with the issue of this journal, the International Testing Association is holding the final meeting of its great International Congress at New York, and in our next number we hope to commence presenting some of the more important papers that relate directly or indirectly to matters appertaining to the uses of Portland cement and to the application of Concrete and Reinforced Concrete.

In this issue we only wish to record that this important American meeting has just taken place, and to express our regret that—as on previous occasions—the organised representation of the British Empire at the Conference has lacked preparation, and that no systematic effort appears to have been made to give these islands the position in the Conference that we as a Power of some importance should hold.

It is perhaps difficult to define where the fault lies as to this lack of systematic preparation for the representation of Great Britain. No doubt it is partly due to our inane apathy towards matters of research, and also to our tendency towards general "muddling through," which is so much favoured in many walks of life; but perhaps it is yet more due to the absence of one or more leaders who can make up their minds to bring together and organise those of our professional men and scientists, whose main occupation or hobby it is to deal with matters of research and testing.

It is, of course, to be regretted that the result of this lack of organisation and forethought is that Great Britain is more and more taking a "back seat" in the councils of the world on matters of science and technology generally, and that even the little research work we do is neither properly nor effectively presented at conferences of this kind, so that it may be understood by other nations. The natural result is, that when questions of international specification arise, our industries are all too often put at a disadvantage, and our commercial prestige suffers.

It is almost ridiculous that, for instance, the record of what is being done in research work in reinforced concrete is not presented to the Congress by some institution like the Institution of Civil Engineers, the Royal Institute of British Architects, or by the Concrete Institute, and in the absence of anything better, the data regarding the history of British research work has had to be compiled by this journal at the initiative and under the supervision of Mr. Edwin O. Sachs, F.R.S.Ed., who was asked to get together whatever information could be obtained, failing its presentation by some great institution. But perhaps such a *contretemps* is both in the nature of things British in this decade and characteristic of the haphazard development of reinforced concrete in this country, as also of many other matters relating to our technical practice and education. There is no department acting as leader among our public authorities, or among our great institutions, and the little good that is done is largely due to the private initiative and generosity of a few enthusiasts or to the efforts of the Press.

CONGRESS WORK.

As already indicated, we hope to commence to record some of the work done at the Congress in our next issue. In this issue we must limit ourselves to part of the report presented to the International Reinforced Concrete Committee of the Testing Association.

The matter presented makes interesting reading.

Only one other set of particulars is given at this stage--that is, the list of some of the notable papers having bearing on this journal's interests that have been read at the New York meeting. This list is given below and indicates the wide scope of utility of the International Testing Association, which is well recognised on the continent of Europe and in the United States, but is scarcely given a thought in this country.

Reinforced Concrete.

Report of the Reinforced Concrete Committee. By F. Schüle, Zürich.

(*a*) Summary of the Scientific Research Work done in Connection with Reinforced Concrete in Germany since 1909, being the report of the Imperial German Council on Reinforced Concrete.

(*b*) Summary of the Research Work done in Connection with Reinforced Concrete in Austria, presented by Professor B. Kirsch, Vienna.

(*c*) On the Investigations in Reinforced Concrete in the United States of America. By A. N. Talbot, Urbana, Ill.

(*d*) Some notable British Tests regarding the Strength of Reinforced Concrete. By Edwin O. Sachs, London.

(*e*) Reinforced Concrete Tests made in Denmark. By E. Suenson, Copenhagen.

(*f*) Research Work done in Connection with Reinforced Concrete in Switzerland. By F. Schüle, Zürich.

Accidents in Building with Reinforced Concrete. By Dr. F. von Emperger, Vienna.

Tests of Continuous Beams in Reinforced Concrete. By H. Scheit, Dresden, and E. Probst, Berlin.

Relation between the Tensile Strength and Elasticity of Concrete under Compression. By Prof. F. Schüle, Zurich.

Strength of Cement.

Standardised Tests for Hydraulic Binding Media by the Use of Prisms. Standard Sand. Report by F. Schüle, Zürich, President of Committee No. 42.

Strength-Testing and the Utilisation of Prisms made of Plastic Mortar. By M. Gary, Berlin-Lichterfelde, W.

The Progressive Increase in the Strength of Cement Mortars. By E. Candlot, Paris.

Constancy of Volume of Cement.

Accelerated Tests for Constancy of Volume of Cement. First Report on the Principal Question. By Bertram Blount, London.

Accelerated Test for Constancy of Volume in Portland Cements. Second Official Report on the Principal Question. By M. Gary, Berlin-Lichterfelde, W.

The Boiling Test and the Volume Constancy of Portland Cement in Dry Storage. By F. Schüle, Zürich.

On the Testing of Hydraulic Binding Media. By J. Bied, Viviers (Ardèche).

FINEST PARTICLES IN PORTLAND CEMENT.

Determination of the simplest Method for the Separation of finest Particles in Portland Cement. By M. Gary, Berlin-Lichterfelde.

Air-sifting as a Method for the Quantitative Determination of the finest Particles contained in Pulverulent Materials. By R. Feret, Boulogne-sur-Mer.

Determination of the very fine Powder in Portland Cement. By Mayntz Petersen, Copenhagen.

The Significance of the finest Portland Cement Flour. By F. Schüle and H. Gottrau, Zürich.

POZZOLANA.

Diatom-Earth as Pozzolana for Cement. By A. Poulsen, Limveg.

CEMENT IN SEA WATER.

On the State of Preservation of Test Blocks and Blocks Proceeding from the Harbour Works, immersed in the Baltic Sea, at Libau Harbour. By W. Czarnomski, St. Petersburg.

Action of Sea-water on Hydraulic Binding Media. By Lombard and Deforge, La Rochelle.

Notes on the Action of Sea-water on Reinforced Concrete. By V. J. P. de Blocq van Kuffeler, Hoorn.

Proposal for establishing a standard SO_3 Content for Portland Cement. By the Verein Deutscher Portlandzement-Fabrikanten, Kalkberge (Mark).

Note on the Comparative Mechanical Strengths and the Stabilities in Sea-Water of Crushed Sand Mortars and Sea-Sand Mortars. By Gassier, of Marseilles.

The Porosity of Mortars. By J. Bied, Viviers.

VARIOUS ON CEMENTS.

On Various Chemical Phenomena encountered in the course of Industrial Investigations. By J. Bied, Viviers.

STONES AND MASONRY.

Weather Resistance of Building Stones. By A. Hanisch, Vienna.

Note on Appliances for Testing Paving Setts. By Pierre Labordère and Frederic Anstett, Paris.

Influence of the Composition of Mortar and the Quality of the Stones upon the Durability of Masonry. By J. A. van der Kloes, Delft.

Contribution to the Question: The Influence of Faulty Mortar Mixtures. By Anton Hambloch, Andernach on the Rhine.

The Physical Testing of Broken Stone Railroad Ballast. By A. T. Goldbeck and F. M. Jackson, Washington.

CONCRETE.

Electrolytic Action on Unreinforced Concrete. By Cyril de Wyrall, New York.

Waterproof Concrete. By Alb. Grittner, Budapest.

Tests for Concrete. By Cloyd M. Chapman, New York.

Tests of Concrete Materials by the U.S. Reclamation Service. By I. Y Jewett, Denver, Col.

FIRE TESTS.

The Fire Resistance of Concrete and Reinforced Concrete. By Edwin O. Sachs, London.

Fire Tests with Floors in U.S.A. By Ira H. Woolson and Rud. P. Miller, New York.

Cements Proving Constant in Practice although having Failed to Pass the Boiling Test. By M. Gary, Berlin-Lichterfelde.

MATERIALS FOR ROAD MAKING.

Principles of Unified Methods of Tests on Road Making Materials. By A. Mesnager, Paris, with supplements:

On the Methods Used in Various Countries.
(*a*) Germany. By H. Burchartz and B. Stock, Berlin.
(*b*) Austria. By A. Hanisch, Vienna.
(*c*) Belgium. By E. Camerman, Brussels.
(*d*) Denmark. By H. I. Hannover, Copenhagen.
(*e*) Hungary. By E. Fock, Budapest.
(*f*) Norway. By Skougard, Kristiania.
(*g*) Netherlands. By M. E. H. Tjaden, Breda.

From the above list of contributions to this great Congress the extraordinary lack of data presented by Great Britain is very noticeable—three papers only out of forty-five on subjects recorded above.

As to the representatives of Great Britain at the Congress, the delegates formally authorised are given below, but unfortunately several of these representatives were prevented from attending at New York, and the delegation—in itself a small one—was consequently reduced to a very small one, and of that reduced delegation no less than five are resident in America.

The German Government, it should be added, had ten delegates, and the Russian Government six.

The British official representation is as follows :—

By the Government:
- Dr. W. C. Unwin, F.R.S., President of the Institution of Civil Engineers, London.
- Dr. Archibald Denny, Leven Ship Yard, Dumbarton.
- Bertram Blount, F.I.C., Westminster, London.

By the Institution of Civil Engineers:
- Dr. W. C. Unwin, F.R.S., President of the Institution, London.
- Alfred Noble, New York.

By the Iron and Steel Institute:
- Sir Hugh Bell, Bart., Northallerton.
- Sir Robert A. Hadfield, F.R.S., London.
- G. C. Lloyd, London.

By the Engineering Standards Committee:
- Leslie S. Robertson, M.Inst.C.E., London.

By the Geological Survey:
- J. Allen Howe, Museum of Practical Geology, London.

By the National Physical Laboratory:
- Dr. W. Rosenhain, Teddington, Middlesex.

By the British Fire Prevention Committee:
- Edwin O. Sachs, F.R.S., Ed., Chairman of the Committee, London.

By the Institution of Naval Architects:
- Stevenson Taylor, President, American Society of Naval Architects, New York.
- Naval Constructor Rear Admiral W. L. Capps, United States Navy.

By the Institution of Mechanical Engineers:
- Captain R. W. Hunt, Chicago.

By Lloyd's Register of British and Foreign Shipping, London:
- James H. Mancor, Lloyd's Register of Shipping, New York.

By the Institution of Mining and Metallurgy, Salisbury House, London:
- F. W. Harboard, London.
- Edmund W. Janson, London.
- Arthur L. Pearce, London.

By the University College, Cork, Ireland:
- Professor C. W. O. L. Alexander.

THE NEW KING'S COLLEGE HOSPITAL.

By ALBERT LAKEMAN.

There can be no doubt that reinforced concrete has many advantages over other forms of construction where large public buildings are concerned, and it is therefore pleasurable to see this method of construction adopted in one of our newest hospitals now being erected in the South of London, and of which details are given in the following article.—Ed.

THIS new building is now being erected on a site covering twelve acres, situated on Denmark Hill, from the designs of Mr. W. A. Pite, F.R.I.B.A., and the estimated cost of the whole scheme is about £300,000. It is interesting to note that reinforced concrete on the Hennebique system is being employed for the construction, and it forms another notable example illustrating the extensive use of this material, which is so eminently suitable for large buildings when economy and fire-resisting qualities are the primary considerations. The scheme is the outcome of a desire to provide an adequate hospital for South London, and it was decided to remove King's College Hospital from the West Central district, which is well served with other hospitals, to that district where it was urgently needed. Some idea of the size of the hospital when completed can be gathered from a comparison with that of St. Thomas's. The six ward blocks of the latter institution, which enjoys such a prominent position on the Thames, represent about the length of the site of the King's College

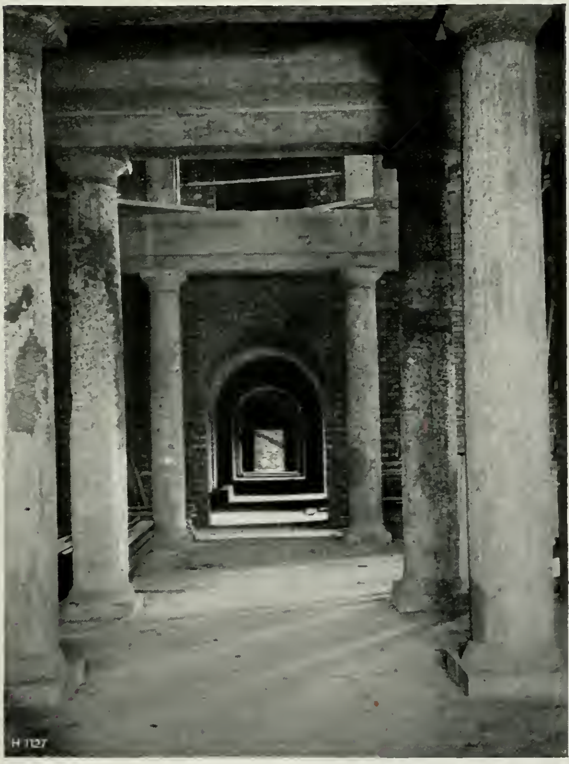


Fig. 10. Concrete Columns in Corridor.
NEW KING'S COLLEGE HOSPITAL.

Hospital, but their depth is only one-half of that which the new building will have. Up to the present only a portion has been completed, and the description in detail is limited to this portion. The administration block, which is now built, stands in the centre, and the other departments are grouped around in accordance with the principles of centralisation and radiation, and the whole of the wards, containing 600 beds, will face the

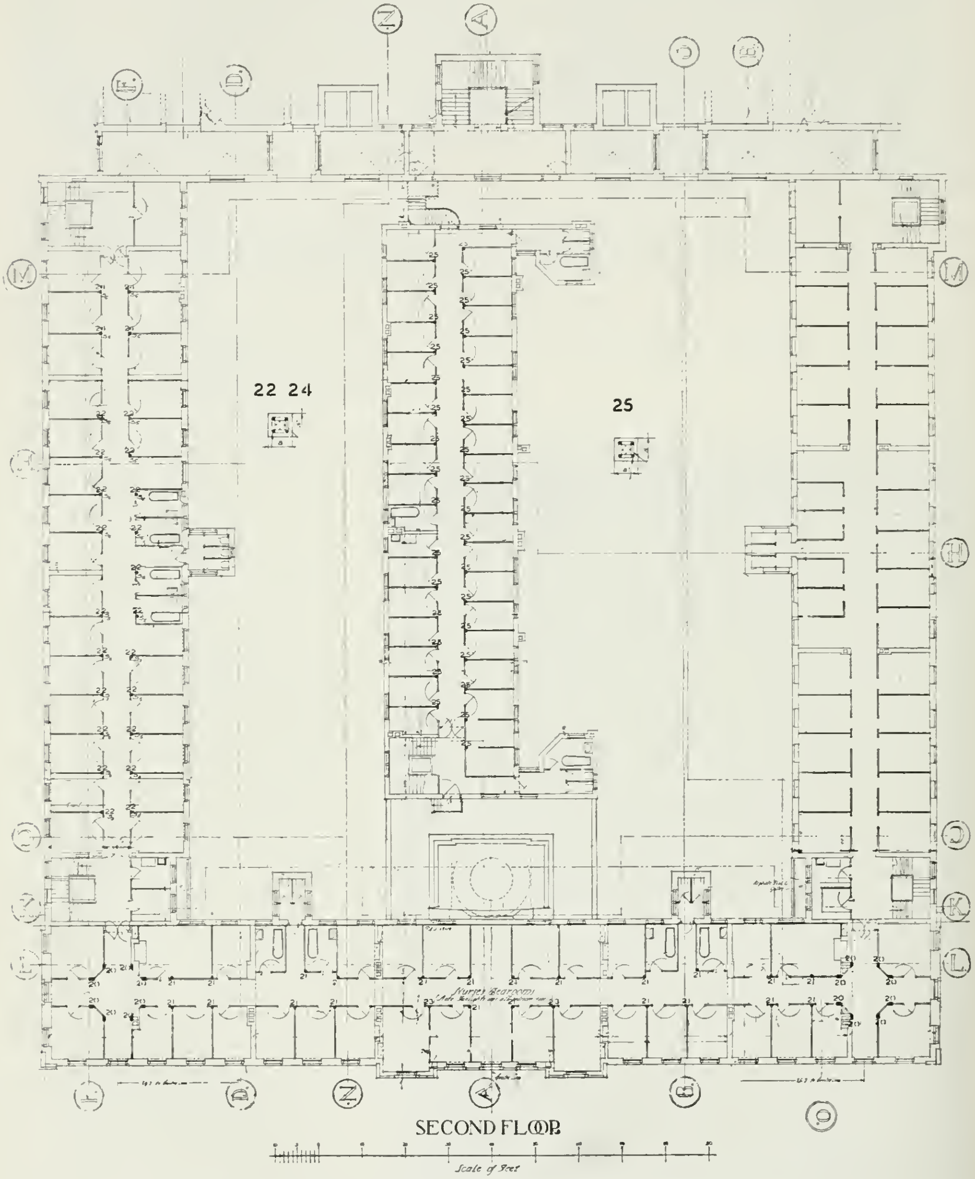


Fig 1. Plan of Administration Block
New King's College Hospital.

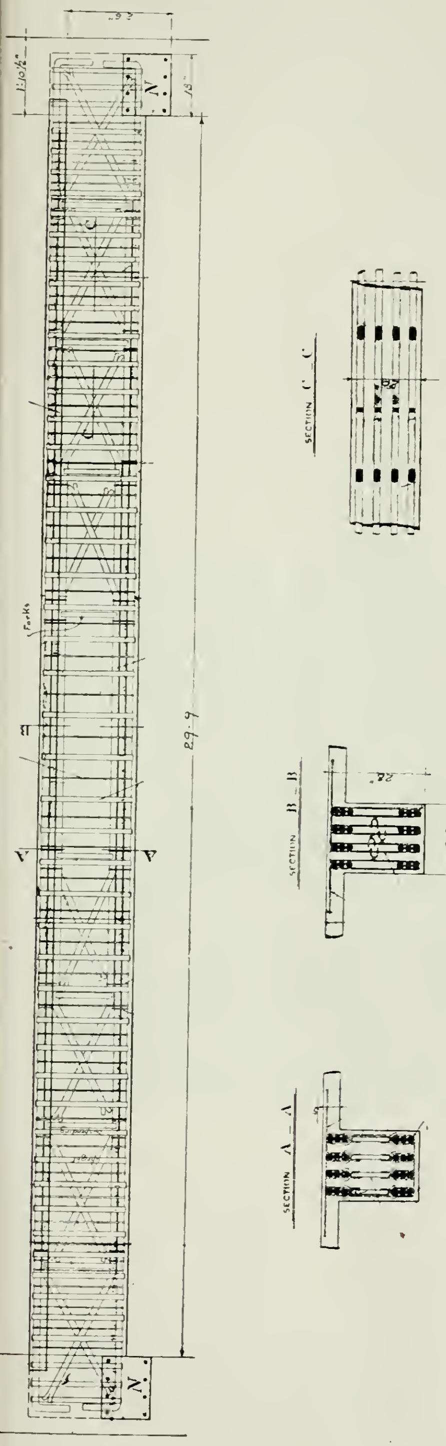


Fig. 2. Beams in Administration Block.
New King's College Hospital.

south. The main spiral corridor is over 700 ft. in length, having the chapel situated at the centre, while behind is placed the underground heating and machinery station, the low roof of which will be utilised as a garden; and the western side of the site will be occupied by the medical school. The central administration block is a six-story building, and this contains the offices and nurses' and servants' home, the general disposition of the building being shown on the plan of the second floor, which is illustrated in *Fig.* 1. The columns in the interior are placed in two rows, one on either side of the corridor at distances of about 9 ft. apart, and the size, generally speaking, is 8 in. square, reinforced with four bars tied with $\frac{3}{16}$ in. links. The type of main beam employed is illustrated in *Fig.* 2, which is a typical example. It has a clear span of 29 ft. 9 in., with a total depth of 28 in. and a width of 20 in. The reinforcement consists of twelve bars on both upper and lower surfaces, these two sets being well wired and kept in position by forks. Stirrups are provided at varying distances for the resistance of the shear, and certain of the upper and lower bars are cranked at the ends and carried through the depth of the beam.

Some interesting work is that constructed below the ground level, as, for example, that occurring under Ward Pavilion No. 5. Here some heavy column and pier loads had to be carried, and as they were close together it was necessary to form a reinforced raft, and this was done in such a manner that the loads were spread over an area exceeding over 50 ft. × 46 ft., giving a pressure on the soil of under 2 tons per sq. ft. The columns were connected by beams

24 in. deep, reinforced with nine or ten bars in the upper and lower surfaces, these beams being in two sets of four, and the two sets at right angles to one another; while the whole area enclosed by the beams was filled in with concrete 24 in. deep and reinforced with $\frac{7}{16}$ in. diam. bars at 8-in. centres in both directions and in both surfaces. An underground tunnel was also formed at this point, sections of which are illustrated in *Fig.* 3. This has a width in the clear of 4 ft. 6 in., and a height of 7 ft. in one portion

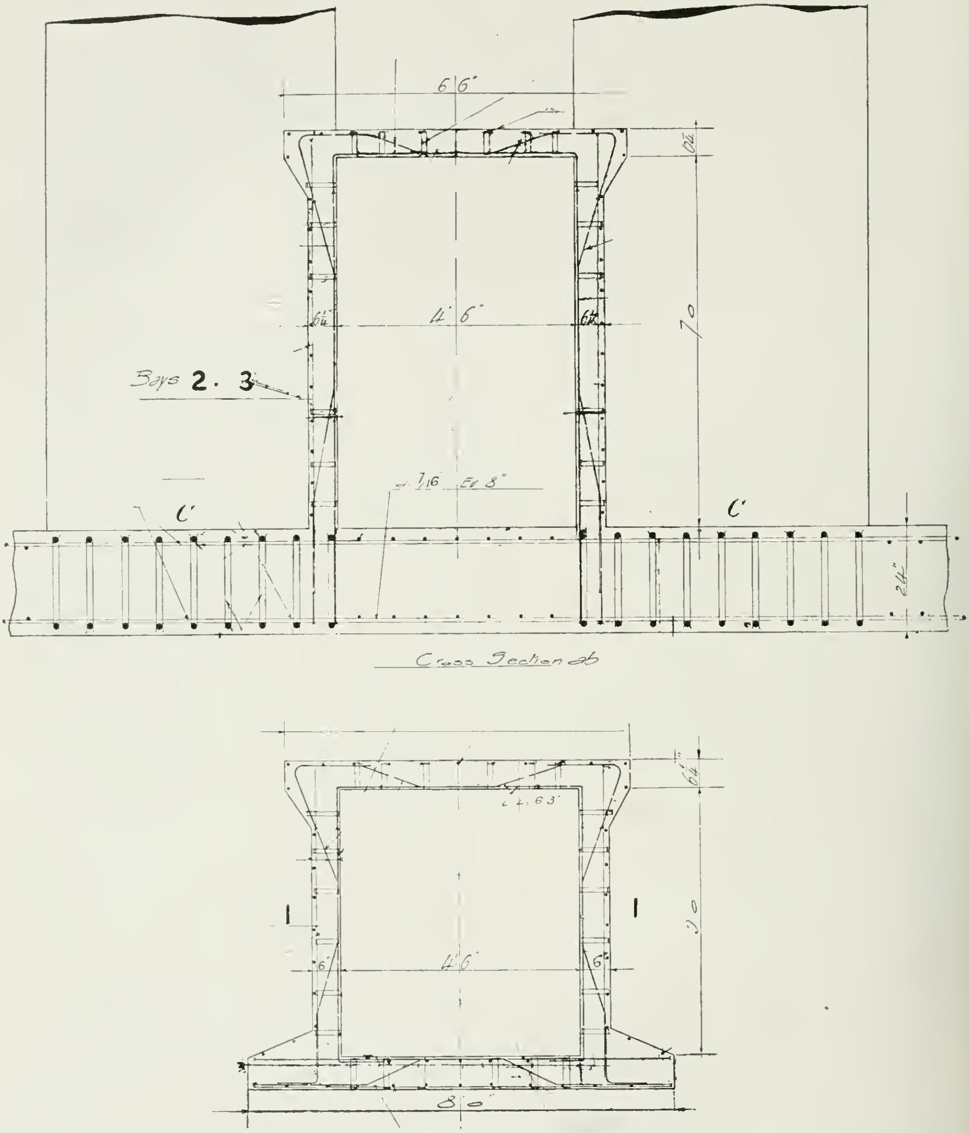


Fig. 3. Sections of Underground Tunnel.
New King's College Hospital.

and 5 ft. in another, the walls and roof having a thickness of 6 in., with reinforcement in both surfaces; while the floor, where not formed by the raft, has a thickness of 9 in. The details in *Fig.* 4 illustrate some further work which comes below the ground level under Blocks 4 and 5, this being a passage-way 6 ft. wide, formed with walls and floor of concrete 5 in. thick, reinforced with $\frac{1}{4}$ and $\frac{1}{2}$ in. bars on both surfaces, with reinforced piers at intervals, which are 14 in. × 9 in., reinforced with four vertical bars well linked together,

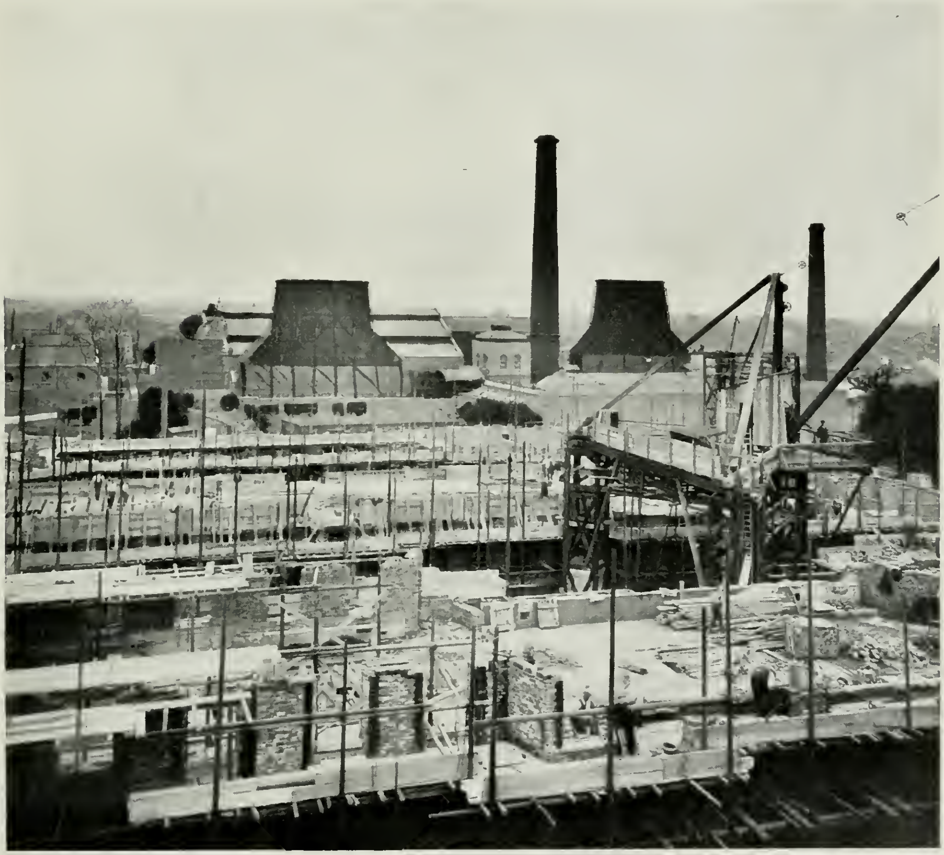


Fig. 8. Ward Block during construction.
New King's College Hospital.

beams being formed below the floor of the passage-way to connect the piers, these beams being 12 in. deep and 18 in. wide. Openings were formed in the upper part of the walls between the piers, as illustrated on the drawings. The diagrams in *Fig.* 5 illustrate the work to the roof of the Ward Pavilion 4 and 5, over which octagonal tank houses are constructed, the latter having a maximum width of about 24 ft. and a height to the springing of the pitched roof of 20 ft. The columns to the tank house are canted on plan, and have a width of 18 in. on either outer face, while the reinforcement consists of six $\frac{5}{8}$ in. diam. bars, bracing being provided by horizontal beams placed at a height of 12 ft. from the tank-house floor level. The solid floor under the tank house is only 4 in. thick, and it is reinforced in both upper and lower surfaces. The type of hollow flooring used is also illustrated, and this is of interest, as it is absolutely essential in a building of this size to provide an efficient construction which at the same time is as light as possible and more or less sound resisting. The Hennebique hollow type consists of hollow tubes formed either of fireclay or concrete, usually measuring about 20 in. wide by 24 in. long, and from 7 to 10 in. high. These are placed in parallel longitudinal rows between secondary beams, the latter being reinforced and moulded in position between the tubes, which are temporarily supported on planks. This system has the advantage that no elaborate centering is necessary, as the sides of the tubes form a permanent shuttering for the vertical portion of the reinforced beams. The top of the tubes are curved slightly to increase the strength, and the sides are inclined slightly outwards towards the top, thus preventing any tendency for the tubes to drop out on the underside of the floor. A minimum covering of 2 in. of concrete is placed over the centre of the tubes, giving a total thickness in this instance of 12 in. A large kitchen is formed at the basement level between two of the wings of the administration block, the main portion occupying a space 60 ft. long and 26 ft. wide, and this has curved roof trusses or beams which rise 10 ft. above the intersection with the vertical wall face. These trusses or beams are placed at about 10 ft. intervals, and have a thickness of 7 in. and a depth varying from 18 in. to 20 in., the reinforcement, generally speaking, consisting

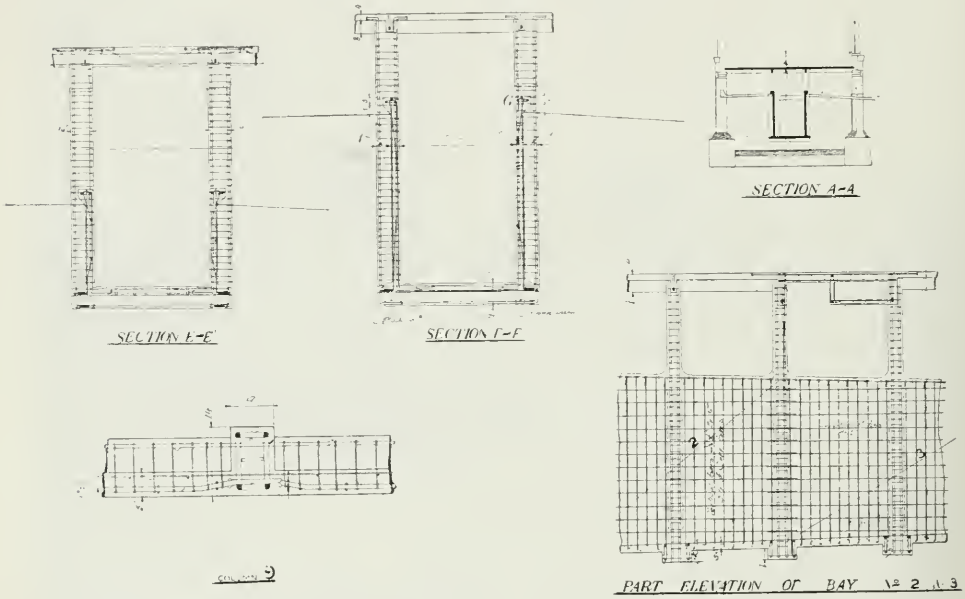


Fig. 4. Details of an Underground Passage.
NEW KING'S COLLEGE HOSPITAL.

of four 1¼ in. diam. bars. The lower part of the roof proper is constructed of concrete 4 in. thick, curved to a radius of 5 ft. 6 in., and this terminates in a vertical portion 4 ft. high, formed with openings, as shown in the drawings illustrated in *Fig.* 6, over which a pitched glass roof is formed. Certain of the trusses are carried by independent columns, which have a free length of about 10 ft., and these are 10 in. × 9 in., reinforced with four bars and links at 3 in. pitch. The ends of two trusses are carried by a beam over a span of 28 ft., and this beam has a depth of about 4 ft. and width of 9 in., and is reinforced with two bars in the upper and lower surfaces. The load from this beam is carried by the columns mentioned above, and these latter have a foundation slab 4 ft. 6 in. sq., with a minimum thickness of 9 in., thus distributing the load in such a manner as not to exceed 2 tons per sq. ft. on the soil. An external duct is formed in reinforced concrete at the foot of the roof on the outside of the building, and projecting 18 in. from the face of the wall, and this only has a thickness of 3 in. The staircases shown on the

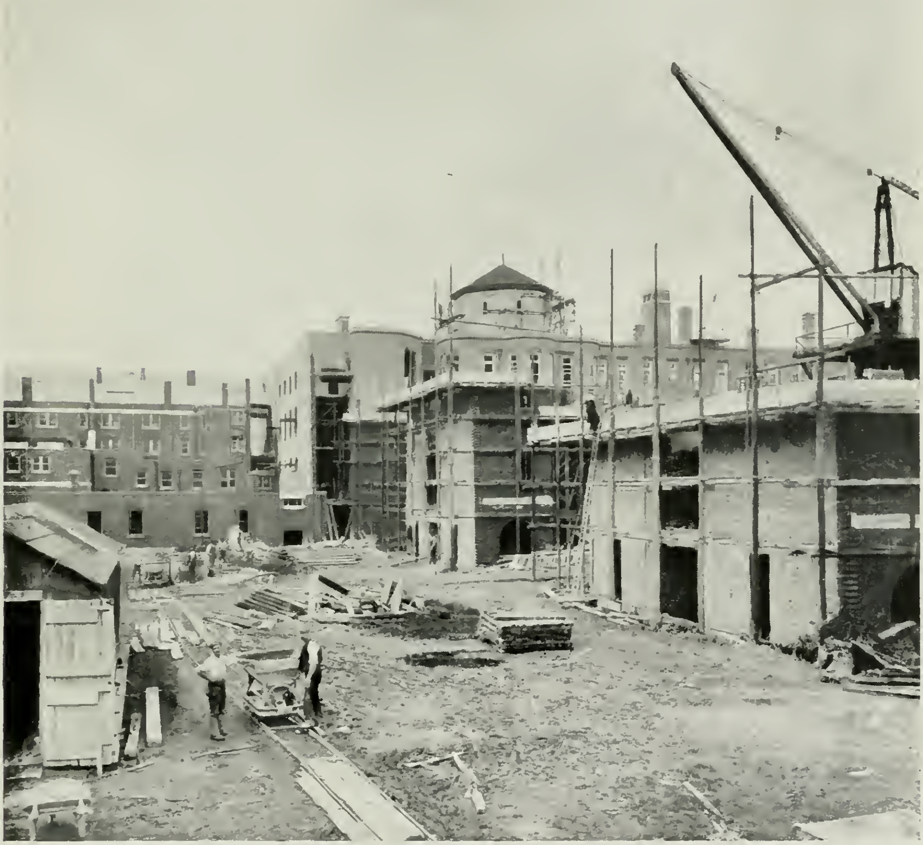


Fig. 9. General View of Buildings.
New King's College Hospital.

general plan, illustrated in *Fig.* 1, are constructed entirely in reinforced concrete, with raking concrete stringer beams and reinforced soffits. One cross-bar is also placed in each tread, and the landings are formed with a thickness of 3 in. reinforced in both directions. There are a great many features of

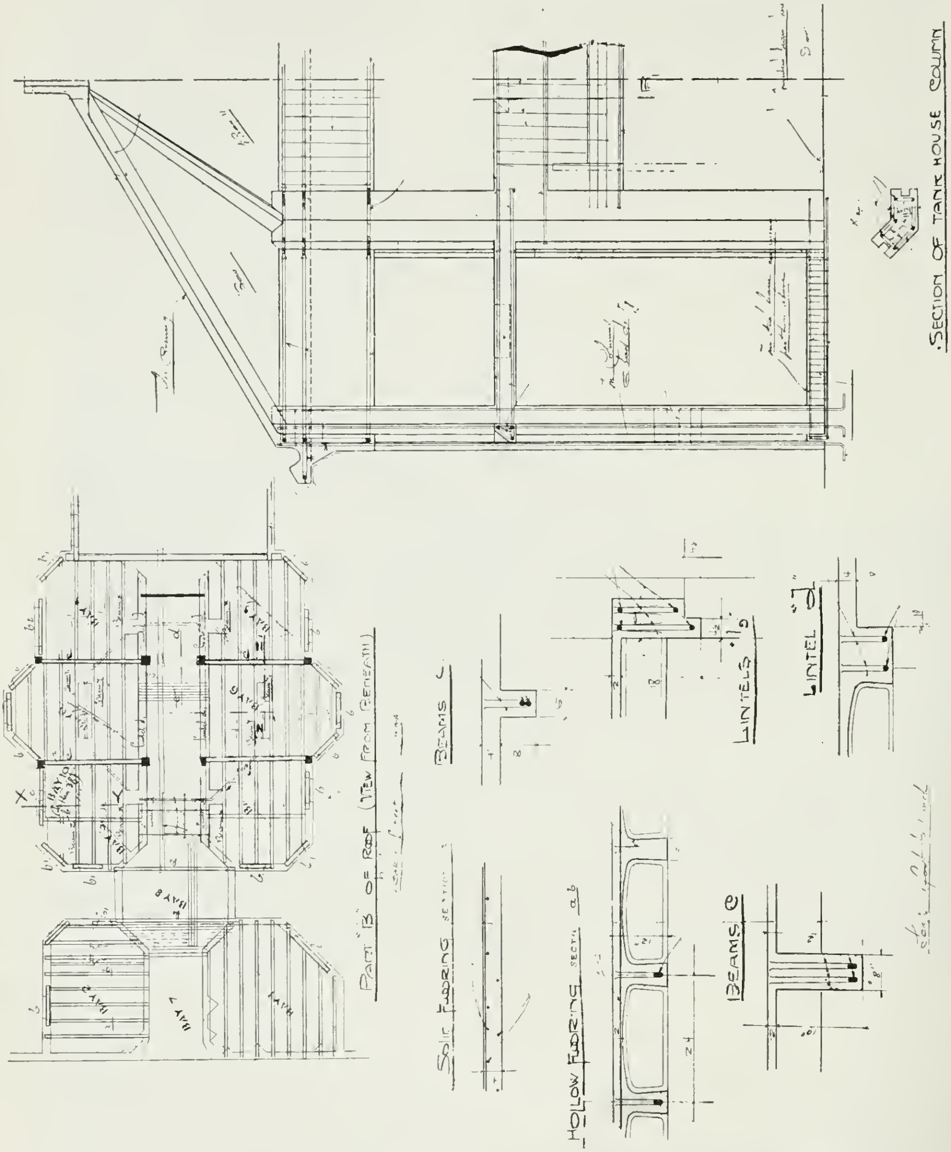

Section of Tank House Column
Part "B" of Roof (View from Beneath)
Beams c
Lintels
Lintel "d"
Solid Flooring section
Hollow Flooring section ab
Beams e
Bay 10
Bay 8
Bay 7
Bay 9

Fig. 5. Tank House, over Ward Pavilions. New King's College Hospital.

constructional interest which cannot be described at present, as the whole building is not complete, but it must be considered as an excellent example of reinforced concrete work; and the adoption of the material for these buildings shows how general the use is becoming for all schemes of importance. Messrs.

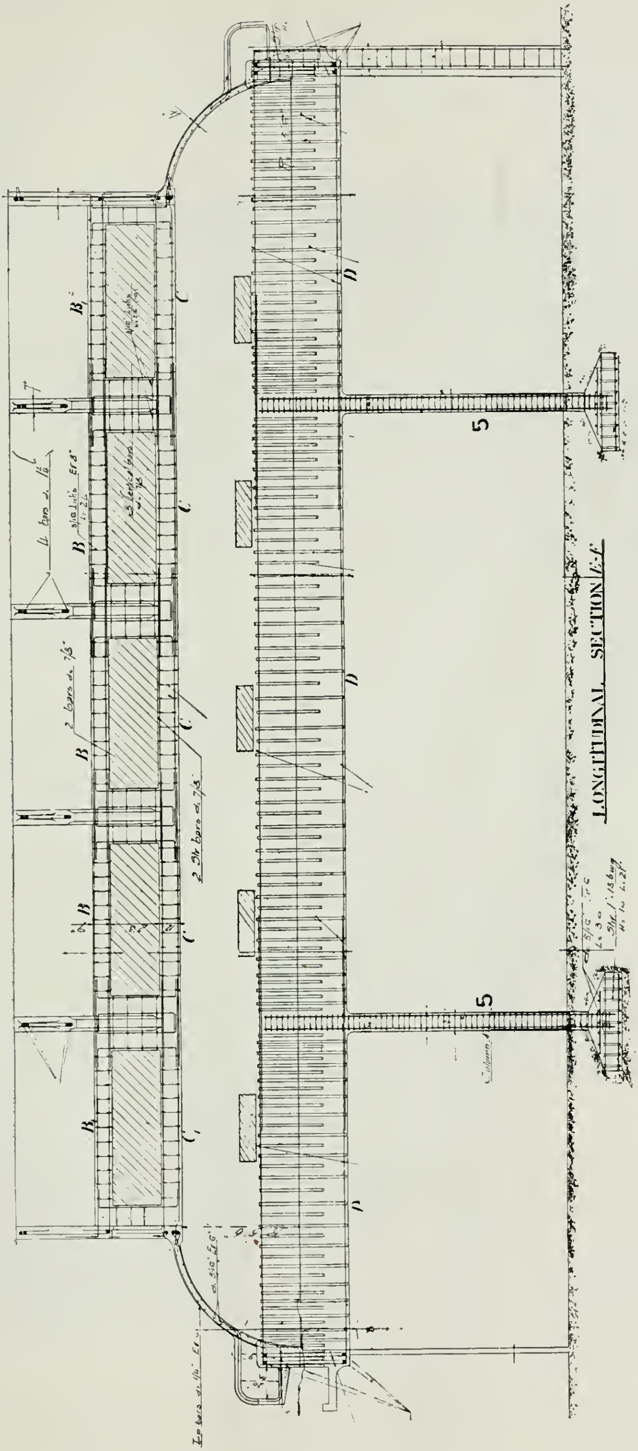


Fig. 6. Section through Kitchen.
NEW KING'S COLLEGE HOSPITAL.

Mouchel and Partners, Ltd., are responsible for the design of the reinforced concrete work, etc.; Professor D. S. Capper, M.Inst.C.E., acting as consulting engineer for the heating and ventilating; while the contractors are Messrs. Foster and Dicksee.

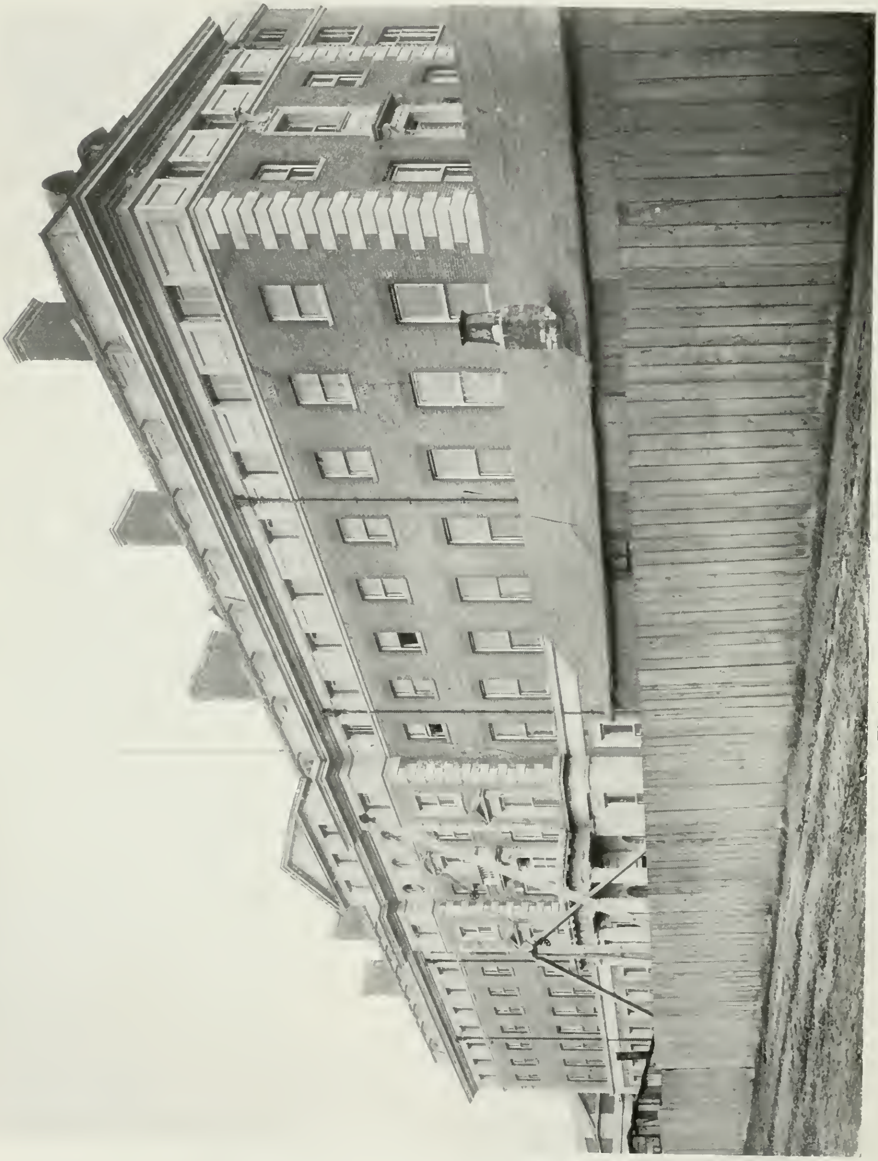
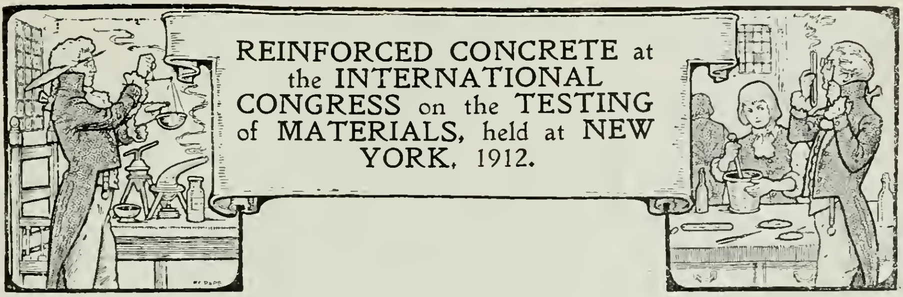


Fig. 7. View of Administration Block.
New King's College Hospital.



REINFORCED CONCRETE at the INTERNATIONAL CONGRESS on the TESTING of MATERIALS, held at NEW YORK, 1912.

The following is the first part of the Report presented by the Reinforced Concrete Committee formed by the International Association for Testing Materials, which Committee has just held its meeting at New York on the occasion of the International Association's Congress. The Report includes Appendices as to Research Work done in different countries, and of these appendices those dealing with the United States, Great Britain, and Denmark are presented herewith in full, whilst those dealing with Germany, Austria and Switzerland, which are only printed in the German language, are presented in the form of summarised translations. The report should be read in connection with some of our earlier contributions on the work of the International Reinforced Concrete Committee.—Ed.

REPORT OF THE REINFORCED CONCRETE COMMITTEE OF THE INTERNATIONAL ASSOCIATION FOR TESTING MATERIALS.

Presented by Professor F. Schüle, Zürich (Chairman of the Committee).

GENERALLY.

Since the Congress at Copenhagen the composition of the Reinforced Concrete Committee has been slightly modified. It now consists of :—

Chairman—F. Schüle, Professor at the Federal Technical High School and Director of the Federal Laboratory for Testing Materials, Zurich.

Vice-Chairmen—W. Gernielmann (Berlin), Professor A. N. Talbot (University of Illinois), U.S.A.

Members—W. H. Warren (University of Sydney); Paul Christophe (Brussels); T. Grut (Danish Royal Engineers, Copenhagen); E. Suenson (Royal Technical High School, Copenhagen); Bürstenbinder (Hamburg); Alf. Hüser (Cassel); Professor Dr. A. Martens (Royal Testing Laboratory, Berlin-Lichterfelde); E. Züblin (Strassburg); Max Clarke, F.R.I.B.A. (London); W. Dunn (Concrete Institute, London); Dr. Dyckerhoff (Amöneberg); Edwin O. Sachs, F.R.S.Ed. (British Fire Prevention Committee, London); Dr. W. C. Unwin (Institution of Civil Engineers, London); A. Considère (late Inspector-General of Roads and Bridges, Paris); R. Féret (Roads and Bridges, Boulogne); Professor A. Mesnager (Government Laboratories); Ch. Rabut (Versailles); M. Tricon (Société des Travaux en Béton Armé, Paris); W. Hanegraaff, Captain (Bussum, Holland); Silvio Canevazzi (Technical College, Bologna); Camillo Guidi (Engineering School, Turin); Claudio Segrè (Railways Experimental Laboratory, Rome); V. Brausewetter (Vienna); Dr. F. von Emperger (Vienna); Professor B. Kirsch (Imperial Technical High School, Vienna); Professor J. Melan (Imperial German Technical High School, Prague); Professor Dr. Belelubsky (Technical Mechanical Laboratory, St. Petersburg); Professor S. Droujinine (Polytechnic Institute, St. Petersburg); Professor N. Abramoff (Don Polytechnic Institute, Nowotscherkask); N. Bogouslawsky (Russian State Railways, St. Petersburg); Hirszson (St. Petersburg); N. Jitkewitsch (Colonel of Engineers, St. Petersburg); R. Maillart (Zurich); Professor Dr. Richard Zielinski (Polytechnic, Budapest); J. Schustler (Budapest); Professor A. Czako (Budapest); Professor E. Turneaure (University of Wisconsin, U.S.A.); L. Richard Humphrey (Philadelphia, U.S.A.).

On the occasion of the Congress at Copenhagen, in September, 1909, there was a plenary sitting of the Committee, attended by twenty-three members from eleven different countries. The principal decision was the adoption of the list of uniform notation, drawn up by the sub-committee under the chairmanship of Professor Melan, for publications in languages other than English. That list is appended to the present report, and the Committee hopes that it will usefully contribute to the unification of the expressions in use, and thus facilitate the editing and study of the literature of reinforced concrete.

For the English-speaking countries a scheme of notaton, drawn up by the Concrete Institute in London, and presented by Mr. E. O. Sachs, exhibited somewhat considerable deviations from the sub-committee's list. The English and American members were invited to revise the English draft, so as to bring it into as close an approximation as possible to the adopted list.

The other deliberations consisted chiefly of a discussion on the progress of the labours of our Committee, with the result that the most important point appeared to be to complete the reports on the course of the experiments in the different countries by presenting separate reports for each country, and then to arrange the results of the experiments systematically.

In a circular-letter addressed to the members of the Committee on February 2nd, 1911, I drew attention to the fact that the reports on experiments which had been undertaken in several countries had not been included in the Proceedings of the V Congress, and that it would be desirable to complete the communications for the period anterior to 1909, adding thereto a summary of the work undertaken since that year.

The investigations will not acquire their full value until the results obtained have been tabulated. This can already be done in the case of the flexion tests applied to beams. Numerous experiments have led to analogous results, and form a basis for specifications on reinforced concrete. Thus, in beams subjected to bending strain up to the point of rupture, the normal case is that in which the rupture is caused by the yield point of the iron being exceeded.

The abnormal cases, which it is most important to observe, are those in which the beam subjected to compression has been crushed before the yield point of the iron (under tension) has been attained; or those in which the rupture has been brought about by shearing under the influence of shearing strains.

Certain valuable information could be afforded in respect of these abnormal cases:—

(*a*) by giving the percentage amount of metal with which rupture commences in consequence of the crushing of the cement in the compression area;

(*b*) or, in connection with beams ruptured by shearing, a statement of the maximum shearing strain of the concrete without reference to the iron, and a statement of the percentage amount of iron—in the form of straight and bent bars and yokes—at the ends of the beams.

NOTE.—For the sake of simplicity, the strain and percentage content should be referred to the useful section of the concrete in beams of rectangular section, or in the web of beams of T-section.

Such values, being independent of the dimensions of the beams, would possess the great advantage of enabling the engineer to form a reliable judgment, even before proceeding to make exhaustive calculations. They would enable him to ascertain whether any important differences are inherent in the quality of the concrete or iron obtained from different countries.

Though, in the case of columns and pillars, the experiments on the action of centric loads have been carried pretty far, too little progress appears to have been made with the experiments on the effect of eccentric loads to enable one to propose, at present, that the chief results obtained should be tabulated.

The members of the Committee were also invited to send in reports, for each country:—

1. On the methods of testing the materials of which reinforced concrete is formed, and of the extent to which this means of control is exercised;

2. On the progress made in the experimental examination of finished structures, and to mention any necessary amplification of the examination of structural parts, chiefly with the object of establishing more firmly the basis for determining the permissible strains, and of ascertaining the influence of age on reinforced concrete, so as to identify the circumstances which are adverse to the perfect preservation of the metal and concrete.

These points were brought forward by Mr. Rabut in his 1909 report.

So far as accidents to reinforced concrete buildings are concerned—valuable sources of information which one would be glad to have no need for—it was impossible to discuss Dr. von Emperger's highly interesting report at the Copenhagen Congress That author has been kind enough to complete his report by recording the most important facts revealed by the statistics for the period since 1909.

It follows from what has been stated above, that the work of our Committee in the period just expired was of a preparatory nature—namely, the collection of the results of experience in each country, a protracted task which can only yield the desired results through the active collaboration of the members of the Committee.

PRESENT POSITION OF THE WORK OF THE COMMITTEE.

Reports in accordance with the programme sketched in the circular letter of February 2nd, 1911, have been received from various countries; and these form material for discussion at the next Congress. It seems, however, useful to mention, briefly and unofficially, the chief characteristics of the period just expired.

1. EXPERIMENTAL STUDY OF REINFORCED CONCRETE.

The matter which specially interests our Association—namely, the study of the properties of reinforced concrete under the influence of external forces; its physical properties under the influence of temperature, contraction in air, expansion under water; the influence of external agencies, such as fresh water, sea water, steam, smoke, sulphurous acid, ammonia, oils, tar, and electric currents—has made considerable progress, mainly through the carrying out of the extensive experimental programme laid down by the German Reinforced Concrete Committee with the support of the State and of the large German Associations. Publications of great interest have appeared or are in preparation, which report on the experiments carried out, in accordance with that programme, at the German State Laboratories for Testing Materials. This extensive masterpiece constitutes the most important fact of the period just elapsed. It testifies to the importance attached in Germany to the new method of building, not only by builders, but also by the Government; and it also gives proof of the intention to establish all the applications of reinforced concrete on a thoroughly secure foundation. These publications will form one of the most important sources for the study of the question, and of information for the future labours of our Committee.

The arrangement of the results has been commenced, but has not yet proceeded far enough to form a basis for discussion, so long as the more important investigations in progress are not nearly completed.

A circumstance favourable to the determination of the percentage of reinforcement beyond which the crushing of the concrete under a compression strain causes the rupture of a beam subjected to bending strain, is the experimental investigation of the reinforced check beam as proposed by Dr. von Emperger.

So far as concerns the conditions which assist beams to withstand the action of shearing strains, these have been exhaustively dealt with in the case of reinforced concrete beams, having a mean compression strength of about 240 kilos per sq. cm. in No. 12 of the publications of the German Reinforced Concrete Committee, in which Professor C. Bach and Mr. Graf reported on the tests performed on reinforced beams at the Stuttgart Testing Laboratory in accordance with the programme laid down by Professor Mörsch and Mr. Stangenberg.

Although the experiments carried out or proceeding in the different countries are fairly complete, there are still many points to be investigated by experiment, since it is a characteristic feature in connection with reinforced concrete that the engineer dealing with this material considers it necessary to test by experiment the conclusions obtained by mathematical deduction, and also to test the accuracy of the hypotheses on which his calculations are based.

2. CHECKING THE MATERIALS.

The checking of the materials used in reinforced concrete comprises in the first place the testing of the individual materials: iron, cement, sand, and gravel. The methods of testing iron and cement have been fixed in each country, and also form the subject of special work on the part of our Association, so that it is unnecessary to go into them further in the present Report. Nevertheless, it will be useful to call to mind that the accelerated test for constancy of volume, and especially the test by boiling at 100°, are of great importance in the case of cement intended for structural work which will be exposed to dry air and protected from rain—the most usual condition in reinforced concrete structures.

The concrete for reinforced concrete structures is almost exclusively mixed in a plastic condition; and a method of testing the binding medium in the state of plastic mortar will enable the binding media to be classified in a rational manner according to the tensile strength they exhibit in practice.

Up to the present our Association has not concerned itself with the methods of testing sand and gravel for the manufacture of concrete. The utility of such tests is becoming more and more recognised; and the determinations which enable the quality of these materials to be estimated are as follows:—

Origin and petrographical character; specific gravity; classification by screening through sheet metal screens with round perforations; volume of interstitial spaces in the dry state.

The usual distinction between sand and gravel (or stone chippings) needs more precise definition, by fixing 5 or 8 mm. as the diameter of the perforations in the delimiting screen. Grains which will pass through this screen constitute sand, whilst those remaining upon it as residue form gravel or stone chippings. The investigations instituted for some years past in the different countries should furnish our Committee with a basis for determining the characteristic properties of a sand or gravel of good quality, especially as regards the permissible proportion of dust (fine sand). It is also advisable to establish a reliable method of taking the samples for testing.

For checking purposes, the only interest attaching to testing the combined materials is in the determination of the crushing strength of the concrete; and even the recently proposed experiments with reinforced check beams serve no other purpose than to induce phenomena of rupture through the crushing of the concrete in the compression area—that is to say, to check the crushing strength of the concrete by an indirect test.

The test pieces in most frequent use consist of concrete cubes, stored, as far as possible, under the same conditions as the concrete of the structure. The dimensions of these cubes vary, in different countries, from 12 to 30 cm. side. The smaller cubes are specially suitable for checking comparatively thin roof work, whilst the large cubes serve for checking the concrete used in large masses and containing gravel or broken stone of about 4 cm. diameter and upward. The influence of the air and the prevailing temperature can be more effectually determined on test pieces whose dimensions correspond favourably with relation to the thickness of the material in the structure.

In addition to the bending strength, the check test for concrete furnishes two crushing strengths for each test piece. The dimensions approximating satisfactorily to practical conditions are 12 by 12 by 36 cm., provided the grains of gravel or stone chippings are not more than 2½ to 3½ cm. in diameter.

Whichever type of test piece be taken, it is important that the testing should not be restricted to single test pieces, but extended to series of three or four test pieces, so that the uniform preparation of the test pieces can also be investigated.

Finally, it is appropriate to mention the chief checking test for concrete: determination of the density. The sp. gr. of the filling material (basalt excepted) varies but slightly, and is about 2·55 to 2·70, whilst that of Portland cement of good quality is nearly constant (3·10 to 3·15). The proportions in which the concrete for reinforced work is mixed are determined, for structures of the same kind, by the Specifications in each country. The density can be checked in lumps of any shape, but these should weigh at least 1 to 2 kilos and be detached from the structure. All the factors which impair the tensile strength of the concrete—except the quality of the cement—also lower its density—*e.g.*, unsuitable ratio of sand to gravel; insufficient proportion of cement; defective ramming. These checks, applied to test pieces taken direct from the structure, will advantageously supplement the particulars furnished by the testing of special test pieces for the strength of the material. The results of the compression tests and the determination of the density can be combined with advantage. In the Zurich Testing Laboratory the density is determined on lumps weighing a few kilos each by means of a special hydrostatic balance. In order to prevent the water from forcing its way into the cavities of the test piece while the volume is being ascertained, coating the test piece all over with a thin layer of plasticine furnishes good results.

Checking the materials, separately and in combination, forms an important factor for the safety of the structures; and though the differences in the test pieces used seem to complicate the matter and hinder elucidation, it is, before everything, important that checks should be applied and as regularly as possible. In the course of a few years experience will show which are the safest and most practical arrangements to adopt. By emphasising the necessity for such checks, and by tabulating the experience gained in this branch, our Association will do excellent service in the matter of the security of buildings.

3. *EXPERIMENTAL INVESTIGATIONS ON FINISHED BUILDINGS.*

The experiments in this case are not tests continued up to the point of rupture, as is the case with laboratory tests, but loading tests with nett loads 1½-2 times the calculated load.

To be of any use such tests should be applied to a sufficient extent of surface, and should comprise the simultaneous reading of the deformations, chiefly bending, at a number of points on the structure. One of the chief advantages of reinforced concrete is that it forms all the parts into a solid whole; but this solidarity can only be clearly revealed by fairly complete loading tests. The contractors and officials on whom the performance of such trials depends, and who have generally to pay the cost of same, do not always thoroughly comprehend the scientific importance of such tests in clearing up our knowledge of reinforced concrete.

The object of our Association is to favour the performance of such experiments, as a supplementary check on the calculations and materials used. In themselves alone, however, such tests are not sufficient to furnish an accurate judgment of the degree of safety of structure, except in the (fortunately rare) instances in which the structure suffers decided injury from the loading test.

ACCIDENTS.

Dr. F. von Emperger has laid a separate report on this question. I beg to communicate the resolution proposed.

RESOLUTION.

With a view to the prevention of accidents and to facilitating the acquisition of the requisite knowledge of the properties of materials, it seems advisable that a uniform system of reporting on building accidents should be adopted in the different countries, in the same manner as is already done in the case of accidents to steam boilers. The Congress expresses a desire that the Reinforced Concrete Committee shall endeavour to organise such an international system of reports and present to the next Congress the reports on building accidents, collected country by country, together with the deductions made therefrom as regards the prevention of accident.

UNIFORM NOTATION.

At the conference of the Reinforced Concrete Committee of the International Association for Testing Materials, held on October 12th, 1908, at Basle, it was decided to facilitate the comparability of the various reports and experimental data relating to reinforced concrete, by agreeing upon a uniform mathematical notation for the most important dimensions occurring in descriptions and statical calculations.

The sub-committee entrusted with this task, and composed of Messrs. Edwin O. Sachs (England), Mesnager (France), S. J. Rutgers (Holland), J. Melan (Austria), and

R. Maillart (Switzerland), Professor J. Melan being the chairman, agreed on the following proposals, which were approved at the meeting of the Committee at the Rathaus, Copenhagen, on September 5th, 1909.

UNIFORM NOTATION.

It is recommended in general that :

(a) Small Roman letters should be employed for longitudinal data and loads per unit length;

(b) Roman capitals should be used for areas and forces;

(c) Small Greek letters should be used for coefficients and for expressing stresses (forces per unit area).

In particular :

l = the clear span of a bearing structure (girder), or buckling length of a column or stay.
l_m = the total length of a girder or column.
h = the effective depth of cross section of a girder or plate—that is to say, the distance of the centre of gravity of the iron reinforcement under tension from the edge of the compression area.
h_m = the total depth.
b = the width of a beam or of the web of a beam.
b_m = the width of the flange of a beam.
h_l = the thickness of same.
y = the distance of the neutral axis from the outer edge of the compression surface.
d = the diameter of the round column.
d_s = the diameter of the spiral winding.
a, b = sectional dimensions of rectangular columns.
l_o = inside width of an arch.
f_o = inside height of an arch.
l = span of an arch, measured at its axis.
f = vertical height of the axis of an arch.
h_c = thickness of arch at the crown.
h_i = thickness of arch at the springings
A = cross sectional area of the iron reinforcement of a reinforced concrete girder.
B = full sectional area of the concrete.
A_t = cross sectional area (sq. cm.) of transverse reinforcement (stirrups) per 1 metre length of girder.
A_l = cross sectional area of the longitudinal reinforcement of a reinforced concrete column.
A_t = cross sectional area of the transverse reinforcement of a reinforced concrete column.
A_r = cross sectional area (ideal) of the spiral reinforcement reduced to longitudinal reinforcement of equal weight.
$\alpha = 100\frac{A}{B}$ = iron reinforcement as percentage of cross sectional area of concrete.
$\alpha_l = 100\frac{A_l}{B}$ = longitudinal reinforcement as percentage of cross sectional area of concrete.
$\alpha_t = 100\frac{A_t}{B}$ = transverse reinforcement as percentage of cross sectional area of concrete.
$\alpha_r = 100\frac{A_r}{B}$ = spiral reinforcement as percentage of cross sectional area of concrete.
J_b = moment of inertia of the effective cross sectional area of the concrete, referred to the neutral axis.
J_a = moment of inertia of the iron reinforcement, referred to the neutral axis.
$J = J_b + nJ_a$ = total moment of inertia.
E_b = coefficient of elasticity of the concrete.
E_a = coefficient of elasticity of the iron.
$n = \frac{E_a}{E_b}$ = ratio of the coefficients of elasticity.
g = load due to own weight of girder structure, per unit length (metre) or unit of surface (sq. metre).
p = working load per unit length or surface.
G = total own weight.
P = total working load, or individual loads P_1 P_2 P_3
N = axial force in a cross section of the girder.
T = transverse force in a cross section of the girder.
M = moment of flexion in a cross section of the girder.
$P_r N_r T_r M_r$ = the corresponding values at the moment of rupture.
σ_a = tensile stress (kilos per sq. cm.) on the iron.
σ_b = compression stress (kilos per sq. cm.) on the concrete.
τ_a = shearing stress on the iron.
τ_b = shearing stress on the concrete.
$\sigma'_a \sigma'_b \tau'_a \tau'_b$ = corresponding permissible stress on iron and concrete.

NOTE.—A partially modified list will be published for the English-speaking countries.

APPENDICES.

ON THE INVESTIGATIONS IN REINFORCED CONCRETE IN THE UNITED STATES OF AMERICA.

Report of Professor A. N. Talbot, Urbana, Ill.

AT Purdue University, Lafayette, Indiana, tests have been carried on under the direction of Professor W. K. Hatt as follows: In 1908 tests were made of combinations of concrete and hollow tile floor systems which showed that the thickness of the wall of the burned terra cotta tile can be added to the width of the concrete rib to determine the resistance of the latter to shear. Other tests were made of beams continuous over two spans loaded at the centre to determine the best method of reinforcing against shearing stresses. In 1909 tests are being made upon a series of arches to the number of nine, ranging in span from 8 to 15 ft., the arches varying in crown, thickness, and rise; and including segmental, semi-circular and elliptic, both plain and reinforced. These arches are tested under different loadings, and extensometers are placed upon the arch-ring. The span is kept constant and the thrust measured by calibrated spring external to the arch. The purpose is to measure the thrust and the deformation of the arch ring under loadings. The arches are finally broken.

Experimental work on reinforced concrete has been carried on in the Laboratory of Applied Mechanics of the University of Illinois at Urbana, Illinois, for several years. Published information on results of the tests may be found in the following bulletins of the Engineering Experiment Station of the University of Illinois: Nos. 1, 4, 8, 10, 12, 14, 20, 22, 28, and 29. A large amount of data of the results of other investigations are being prepared for publication. Of the work in progress the following summary is given:—

Columns. — In columns reinforced with longitudinal rods, a series with the reinforcement ranging from 1 per cent. to 8 per cent.; a series tested under repetitive loading; and a series reinforced with structural shapes with riveted latticing. In columns with spiral reinforcement, a series to determine the effect of length of column (lengths up to 20 ft.), of pitch of spiral, and of repetitive loading. In columns reinforced with a combination of spirals and longitudinal rods, a series to determine the effect of amount of longitudinal reinforcement, the latter ranging from 2 per cent. to 8 per cent.; also tests to determine the effect of repetitive loading.

Beams. — The investigations recently taken up are for the purpose of determining the effectiveness of various forms of web reinforcement.

Wall Footings. — A series of tests on sections of wall footings tested with a uniform pressure on the bed.

Pier Footings.—A series of tests on pieces made to represent reinforced concrete pier footings of various types and methods of reinforcing. These are tested with an approximation to a uniform pressure.

Recent tests on reinforced concrete at the University of Wisconsin, Madison, Wisconsin, carried on under the direction of Professor F. E. Turneaure are as follows:—

Columns. — During the summer of 1908 crushing tests were made upon plain and reinforced concrete columns. Twenty of these columns 10 ft. long of octagonal or square cross-section (area 118 to 144 sq. in.) were made of 1 : 2 : 4 broken stone concrete. The reinforced columns consisted of four series, reinforced as follows: 1, Structural steel column composed of four angles and lacing bars; 2, Longitudinal rods; 3, Spirals; 4, Longitudinal rods and spirals. The purpose of these tests was to determine the relative strength and elastic properties of these different types of columns. Most of the reinforced columns had a protective shell of concrete outside of the reinforcement. The strength of this shell and the time at which it failed were also studied.

This year tests on spiral-reinforced columns without protective shell are being carried on. There will be made thirty-two columns 8 ft. 6 in. long and 10 in. in diameter. The variables in this series of tests are percentage of longitudinal reinforcement. Duplicate columns will be made to cover every point in each of the variables studied. Crushing tests will be made upon these columns, and their strength and elastic properties studied with reference to their efficiency as structural elements.

T **Beams.** — Tests are also being made from time to time to determine the amount of web reinforcement needed in T beams. A thesis on methods of connecting tension bars in reinforced concrete is being carried on. In addition to the above, some tests on the strength of the flat arch, of plain and reinforced concrete, have been made, and others are under way.

Investigations, now being carried on at the Structural Materials Testing Laboratories at the U.S. Geological Survey under the direction of Richard L. Humphrey, may be summarised as follows :—

Reinforced Concrete in Flexure.—Tests have been made at 4, 13, 26, and 52 weeks on 336 reinforced concrete beams of 1 : 2 : 4 gravel, granite, limestone, and cinder concrete of medium consistency. The reinforcement consisted of $\frac{1}{2}$-in. round mild steel rods and varied from a percentage of 0·5 to one of 2·0.

Tests have been made at 4, 13, 26, and 52 weeks on ninety-six reinforced concrete beams of 1 : 2 : 5 limestone concrete of medium consistency in which the reinforcement varied from 0·5 to 2·0.

Tests have been made at twenty-four hours and one and two weeks on 108 reinforced concrete beams of 1 : 2 : 4 granite, gravel, limestone and cinder concrete of medium consistency. One-half inch round rods of mild steel and three percentages of reinforcement, 1, 1·5, and 2·00 were used. Information regarding the behaviour of reinforced concrete at early ages will be obtained.

Tests have been made at the ages of four and thirteen weeks on 108 reinforced concrete beams of 1 : 1$\frac{1}{2}$: 3, 1 : 2 : 4, and 1 : 3 : 6 gravel and limestone concrete of medium consistency in which the percentage of reinforcement was 2, 3, and 4.

Tests have been made at the ages of four and thirteen weeks on 216 reinforced concrete beams of 1 : 2 : 4 and 1 : 3 : 6 gravel and limestone concrete of medium consistency on spans of 6, 8, and 10 ft. The percentage of reinforcement was 1, 2, and 3.

Tests were made at the ages of four and thirteen weeks on ninety-six reinforced concrete beams of 1 : 3 : 6 gravel and limestone concrete having 3 per cent of reinforcement in three 1-in. round rods, the middle rod being turned up outside the load points. These beams were made in groups by three contractors and the Structural Materials Laboratories' force. Beams were moulded outside in wooden moulds and in the laboratory in steel moulds, both machine-mixed and hand-mixed concretes being used. The information obtained will determine the relation between outside practice and laboratory tests.

This series consisted of forty-eight beams, and is similar to the one above except that but one aggregate gravel was used in a 1 : 2 : 4 proportion and all concrete was machine mixed.

The Structural Materials Laboratories are also now carrying on investigations on reinforced concrete in compression, shear, tension, etc., which are expected to have a bearing upon the action of reinforced concrete work.

Tests have been made at the laboratory of Watertown Arsenal under the immediate direction of Mr. James E. Howard, and the published results are to be found in the annual reports of "Tests of Materials."

Columns.—Most of these tests have been made upon concrete columns, plain and reinforced. The reinforced columns tested have a variety of forms of steel bars for the longitudinal reinforcement. In the spiral reinforcement variations were made in the size of the wire and in the pitch of the spirals. The tests were planned to show the effect of the different styles of reinforcement on the ultimate strength of the column and also to find the time when the spiral reinforcement seemed to come into play. The series was intended to give information on which to base a comparison of the strength and general behaviour of columns reinforced with varying amounts of longitudinal and spiral reinforcement and also of columns without reinforcement and made with richer concretes.

It also should be stated that investigations along various lines are being conducted at a number of other places in this country. The writer has not sufficient information to report upon these, but it is believed that the foregoing statements will give a conception of the nature and extent of the investigations in reinforced concrete being conducted in this country.

SOME NOTABLE BRITISH TESTS REGARDING THE STRENGTH OF REINFORCED CONCRETE.

Some Notes by Edwin O. Sachs, F.R.S.Ed., London.

THE journal *Concrete and Constructional Engineering* has, at my request, collected particulars regarding a number of tests carried out in Great Britain as to reinforced concrete. These particulars were summarised under my supervision and published from time to time as a series of articles. I endeavoured to have this collection made exhaustive, but there were naturally a few experiments of which particulars could not be obtained, owing to their having been conducted by private persons who do not seem inclined to divulge the information. The data were on that occasion presented with little comment, it being thought desirable to place the facts on record in a simple way so that the reader could draw his own conclusions if he so desired, but as a recapitulation of the results obtained is impossible in a short report such as this one, I think a useful purpose will be served by here indicating the more notable tests and the lessons I hold they taught, so that the members of the International Testing Association may obtain some idea of the nature of the work done in the British Isles.

That very little has been done in this country in the way of research work regarding reinforced concrete is, of course, well known. We cannot aspire to having any leading position in this direction, except in respect to the question of fire resistance, which does not quite come within the scope of this report. What little has been done is due to private enterprise alone and there has been no effort to guide these private workers, so that their efforts could be properly collated, compared and eventually form part of some general scheme of investigation. The private individuals and firms, however, who have undertaken tests deserve commendation, for they have performed a service to the community, and particularly to the engineering and architectural professions, which we had no right to expect. Our public institutions, who should have assisted in experimental work, have, unfortunately, been very remiss, for there is practically nothing to place beside the elaborate and highly commendable researches carried out by engineering professors in public laboratories in the United States, Germany, and France. Even our Colonies have done something in this direction, though the Mother Country has omitted to do anything. The only exception, as indicated above, that stands to our credit are the official tests carried out upon the fire-resistance of special systems of reinforced concrete construction conducted by the British Fire Prevention Committee. These tests are especially noteworthy, as very little has been done elsewhere in this direction; the United States being practically the only other country that has carried out any systematic series of such experiments. It is to be hoped that the results of modern investigation in this direction may also some day be recorded for the benefit of the Association.

Floor Slabs and Beams.—Firstly, I would remind the members of the Association of some early tests carried out in England from 1870 to 1876 by Mr. D. Kirkaldy and Mr. Thaddeus Hyatt. They demonstrated the value of anchoring the ends of bars in flat slabs because it was evident that the adhesion failed in certain cases. They also proved the value of fixed stirrups or vertical members in concrete beams, though, due to the load being applied centrally, the value of these vertical members was more in the direction of a mechanical anchorage to the main tension member than in resisting diagonal tension.

The next experiment—*i.e.*, one by Mr. Frederick George Edwards, carried out by Mr. D. Kirkaldy in 1891—also showed the advantage of anchorage to the ends of bars, and that the smaller the reinforcement the less need was there for the anchorage because the perimeter of the smaller bars as compared with larger bars was greater.

In 1893 or 1894 another experiment was carried out for Mr. Edwards which showed that beams could be satisfactorily constructed with diagonal members with holes through the webs.

The tests by Stuarts Granolithic Company in 1893 and 1894 showed some remarkable results with flat slabs heavily reinforced, and can only be explained on the assumption that the stress strain curb on the concrete was more than a parabola, being almost rectangular.

The tests by Sir John Fowler and Sir Benjamin Baker in 1896 on slabs reinforced with expanded metal, also showed remarkable resistance, but insufficient data regarding the strength of the concrete precludes us from drawing any special conclusions therefrom; the same remark applies to the tests by Mr. James Mansergh in 1898, and by the Northern Architectural Association and others in 1899 on slabs also reinforced with expanded metal. The test by Mr. A. T. Walmisley, Mr. J. P. Allen and Mr. A. H. Morton in 1903 was on a large span flat slab, but nothing much can be gained from it except that the ordinary methods of calculation are satisfactory.

Other tests which we published might be examined in detail to show the application of the formulæ in practice, and the factor of safety obtained from the ordinary assumptions, but they contained no special lessons.

Messrs. Hodkin and Jones in 1905 proved how the resistance of a concrete floor was materially increased by the insertion of corrugated bars of some depth extending well up into the floor, and at the same time it was shown how the resistance of ordinary rolled steel joists was increased by the concrete surrounding them in a floor of the old-fashioned type.

The tests by Mr. Blackadder substantiated the adequacy of the ordinary method of calculating rectangular beams in reinforced concrete; those built with shear rods, however, do not show very remarkable results, possibly because of the form of the shear members, which were inclined but not rigidly attached to the main bar, and possibly because of the manner of loading and of the form of the test pieces.

The tests conducted by Messrs. D. Kirkaldy and Son in 1902 for Messrs. A. E. Williams are interesting as being in respect to doubly reinforced rectangular beams.

The tests on the Herbst tubular floor constructed by the Armoured Tubular Flooring Company, Limited, and conducted by the British Fire Prevention Committee in 1907, showed that there was ample resistance, when carefully constructed, in such floors of moulded ribs and moulded filling pieces which serve as permanent centres for a topping of concrete, such floors being in reality of T-beam construction with a suspended level ceiling.

Columns.—The test by Messrs. D. Kirkaldy and Son in 1907 on a column reinforced only with Kahn bars showed that the inclined ribs of the Kahn bar bent inwards to the centre of the column afforded considerable increase of resistance. The test by the same firm of some arched roof trusses at Hammersmith Baths reinforced with Kahn bars, though it is difficult to draw any detailed conclusions therefrom, proves the strength of such a structure in reinforced concrete.

The tests on columns for H.M. Office of Works, which were also conducted by Messrs. D. Kirkaldy and Son, show that the ordinary formulæ fit the conditions of practice well. It was an interesting fact the columns broke near one end, and an explanation of this is needed.

The test on hooped columns which was carried out some years ago for Mr. William Dunn agreed generally with the results obtained by Professor Talbot, considerable deformation resulting before the hooping apparently came into play, and this was recently criticised as resulting in the concrete being overstressed in the early stages and the column deformed, before the hooping proved effective, which it eventually did, in increasing the ultimate resistance of the column. The inference drawn was that the strength of the column which could be worked upon in practice was merely that of plain concrete, and though correct for a column so made was not applicable to columns in which a spiral binding was used together with a fair amount of vertical rods, which combination has been shown by other tests not to have considerable deformation in the early stages of loading, this meaning that the concrete was not overstressed, and such columns have greater resistance than plain concrete.

Various.—Various other minor tests went to show, in a general way, the uniformity and general reliability of reinforced concrete, even if they do not add much to our knowledge of theory, or if they cannot be considered as sufficiently detailed to be of scientific value.

The amount of research work which has been done in this country is, however, as indicated above, very small, and it is to be greatly regretted that England, who was the mother of several of the earliest inventors, and in particular of Mr. W. B. Wilkinson, who should be called the real inventor of reinforced concrete as a method of building construction, should be so behind other countries in the investigation of the subject.

It is probably the unscientific manner in which we went about the investigation of reinforced concrete that led to it being undeveloped here and firstly exploited abroad, and finally returning to us when it was regarded as a foreign system. This is too often the tale of invention in England, and our engineering institutions and our professors of engineering have nothing to compliment themselves upon. At the present time we are, in fact—except in respect to the matter of fire resistance—very much behind all the other great nations in our tests on structural materials and systems of construction, which are few and far between, and are only executed on a small scale.

It seems almost hopeless to expect our Government or our engineering colleges—where so often lack of interest or understanding of such problems has been shown—to now pay any attention to the matter, so that we can only hope for the efforts of private bodies such as the professional societies intimately concerned.

Should this slight contribution to the subject, by its poverty compared with the contributions from other countries, assist in an awakening of my colleagues at home as to the necessity of systematic research work, then these lines will not have been written in vain.

REINFORCED CONCRETE TESTS MADE IN DENMARK.

By Professor E. Suenson, Copenhagen.

SINCE the report to the congress held at Copenhagen in 1909 was given, the following tests have been made, Nos. 4 and 5 by Professor Ostenfeld, the rest by the writer.

Beams.— 1. Tests of beams, $2\frac{5}{8}$ in. in height, $3\frac{9}{16}$ in. in breadth, 5·9 ft. in span length and reinforced with 1·5—2·3—3·2—3·9—4·5—6·4—10·4—13·9 per cent. steel. Such eight beams were made of a very weak concrete (1 : 2 : 3, only seven days old), and eight similar beams were made of a very strong concrete (neat cement, five and a half months old). The breaking load P (in the middle of the beam) and the extreme fibre stresses is printed in the following table ($n = E_s : E_c = 15$):

Reinforcement		Round bars $\frac{9}{32}''$ in diameter					Round bars $\frac{9}{16}''$ in diameter		
		2 pieces	3 pieces	4 pieces	5 pieces	6 pieces	2 pieces	3 pieces	4 pieces
Steel area	sq. in.	0·12	0·18	0·24	0·30	0·36	0·48	0·72	0·95
	per cent.	1·5	2·3	3·2	3·9	4·5	6·4	10·4	13·9
Weak concrete	P lbs.	519	577	501	557	602	578	608	625
	f_c lbs/sq. in.	2605	2660	2270	2370	2280	2380	2750	2930
	fs ,, ,,	42000	32600	22250	19900	17400	13700	10700	8740
Neat cement	P lbs.	751	1040	1285	1503	1894	2002	2402	3342
	f_c lbs/sq. in.	3840	4840	6270	6950	8000	8560	11420	14600
	fs ,, ,,	61500	58500	58800	55700	37400	47900	43000	44000

In the first row of beams the yield point of the steel is not reached, not even in the beam with 1·5 per cent. steel, therefore f_c is nearly the same for all the beams. In the second row, on the contrary, the yield point of the steel is reached in all the beams, and, of course, f_c increases with the steel percentage; even in the beam with 13·9 per cent. steel the compressive strength of the cement is not reached.

Beams and Cubes.— 2. Tests of sixty cubes and fifty beams made of twelve various kinds of concrete.

All the beams were reinforced with about 1·65 per cent. of steel, and the breaking load was about the same in all cases in agreement with the yield point of the steel and independent of the quality of the concrete (the compressive strength of the cubes varied from 1,800 to 3,020 lbs. per sq. in.). Beams for concrete testing must therefore have a greater steel ratio.

3. Tests of twenty-five cubes, having a length of side of $7\frac{7}{8}$ in., and 100 beams, $2\frac{3}{8}$ in. in height, $3\frac{9}{16}$ in. in breadth, 5·9 ft. in span length, reinforced with from 1·59 to 42·9 per cent. steel for finding out the necessary steel ratio in beams for concrete testing.

The tests showed that the breaking moment and f_c increased with the steel percentage until about 4 per cent. steel has been reached; when the steel ratio was greater, both the breaking moment and f_c remained constant. The compressive strength of the cubes (twenty-eight days old) was found to be 3,360 lbs. per sq. in., and f_c was found to be 1·26 times as high (when $n = 15$). In beams for concrete testing, the steel percentage, therefore, must be not less than about 6, as the concrete is often stronger than 3,360 lbs. per sq. in. The reinforcement was bars of mild steel, and the yield point may have been situated at about 39,800-42,700 lbs. per sq. in. If steel with a higher yield point is used for the beams, the steel ratio may be lower, because 6 per cent. steel with the yield point 40,000 lbs. has the same effect as 4 per cent. steel with the yield point 60,000 lbs.

4. Professor A. Ostenfeld has made tests on thirty-six cubes ($7\frac{7}{8}$ in.) and thirty-six beams, $2\frac{3}{4}$ in. in height, $3\frac{9}{16}$ in. in breadth, 5·9 ft. in span length, reinforced with from 5 to 7 per cent. steel. He found that when the compressive strength of the cubes varied from 3,660 to 540 lbs. per sq. in., f_c varied not in the same ratio, but only from 1·45 times to 1·73 times the compressive strength of the cubes.

5. By some other tests on twenty-five cubes ($7\frac{7}{8}$ in.) and thirty beams, of which fifteen were $3\frac{15}{16}$ in. in height, $2\frac{3}{4}$ in. in breadth, 5·9 ft. in span length, reinforced with 5·5 per cent. steel, and fifteen were $2\frac{9}{16}$ in. in height, $3\frac{9}{16}$ in. in breadth, 5·9 ft. in span length, reinforced with 5·6 per cent. steel, he found that the higher shape always broke in consequence of shearing stresses.

Gravel.— 6. Tests for finding out the influence of the greatness of the grain of gravel on the compressive strength of the concrete were also undertaken.

REINFORCED CONCRETE TESTS IN GERMANY, AUSTRIA, AND SWITZERLAND.

A further series of reviews of the research work in connection with reinforced concrete which has been carried out in Germany, Austria, and Switzerland respectively since the Copenhagen Congress of 1909 have been published in the form of German Appendices to the Report of the Reinforced Concrete Committee, and of these we have prepared a summary which reads as follows :—

Germany. — The report deals first with the very extensive series of experiments planned in 1909, and executed under the auspices of the German Reinforced Concrete Committee. The laboratories engaged were those of Gross-Lichterfelde, Stuttgart, Dresden, and Darmstadt, and the investigations embraced the influence of many factors on strength, resistance to slipping, expansion, and other properties on both plain and reinforced concrete. Another series dealt with the electrical conductivity of concrete, and with the action of continuous and alternating currents on reinforced concrete and on the adhesion between steel and concrete. Fire tests were also made with two small huts constructed with different aggregates.

For the control of work during erection crushing tests on plain concrete cubes of 30 cm. side are now in use, but experiments are in progress in order to determine how far test beams may be used for the purpose.

In addition to this programme many investigations have been conducted in the testing stations of Brunswick, Darmstadt, Dresden, Gross-Lichterfelde, Munich, and Stuttgart. The loading tests at Gross-Lichterfelde dealt especially with hollow floors, in which the spaces were formed by bricks, terra-cotta channels, wire spirals covered with roofing felt, pumice concrete tubes, and metal tubes, the spans being from

550 to 785 cm. Other floors tested were reinforced by wire netting covered with felt, reinforced tiles, and floor lights of glass held in reinforced concrete frames. The experiments on columns with different forms of reinforcement have been recently published, whilst some novel experiments dealt with the tensile strength of concrete cylinders of different composition, with and without the application of internal pressure. Stairways of reinforced concrete and reinforced artificial stone slabs, of six steps each, were tested by loading the four upper steps and measuring the deflections. Fire tests were also made with concrete as against granite, and also with concrete made with limestone aggregate.

The Stuttgart experiments include some on the influence of granulated blast-furnace slag on the strength of concrete, and many on the influence of composition on strength and resistance to slipping. A series of tables attached contain particulars of beams tested by bending in which fracture occurred, (*a*) by crushing of the concrete under compression before the elastic limit of the steel under tension was reached, (*b*) by rupture of the beam under transverse forces. All the results have been published, but are newly collected for comparison.

Austria.—The Austrian tests recorded include: Tests of Hennebique beams 9·5 metres long, conducted at Lemberg; an extensive series of compression and bending tests in Vienna; an investigation of the effect of repeated loading on the adhesion of clean and rusty steel to concrete, also in Vienna; and tests of 88 reinforced columns, conducted at Lemberg. The Austrian Association of Engineers and Architects has also undertaken an important series of tests in Vienna, comprising 52 square columns with Portland and slag cement under different conditions of setting and loading, 24 hexagonal columns, and 24 columns with special forms of transverse reinforcement, or 100 columns in all of either 3 or 4·5 metres height; also a large number o. slabs and beams. The influence of composition has been tested by experiments with plain concrete beams. Dr. von Emperger's experiments with columns reinforced with **T** and channel irons are also included.

Switzerland.—The earlier Swiss experiments have served as the basis of new rules for the use of reinforced concrete, adopted in 1909 for all public works with the exception of railways. Investigations have been continued, and have been facilitated by the addition to the Zürich laboratory of a 500-ton Amsler press capable of testing columns 3·8 metres long.

The tests undertaken have included: The effect of reinforcement in the compression zones of beams under bending load; the effect of thickening on reinforced concrete slabs; the effect of different modes of fixing at the ends of beams; a comparison of round bars with Kahn bars; and some further tests on the influence of reinforcement of variable section.

Tests on the components of concrete, all conducted at Zürich, have included: the influence of the form of rectangular test pieces on the crushing strength, and of a number of factors on the strength of prisms, whilst the tests on different Swiss sands and gravels have been continued. Many tests have also been carried out, in accordance with the new regulations, on finished structures, furnishing useful data for comparison with the laboratory results. During 1909 and 1910 the laboratory has tested 1,227 series of concrete test-pieces sent from different parts of the country.

The report concludes with a table, similar to that annexed to the German report, collecting the instances observed of beams breaking (*a*) by crushing, (*b*) by shearing.

PROBLEMS IN THE THEORY OF CONSTRUCTION.

WIND PRESSURE ON ROOFS.

CAONSIDERATION of STANTON'S EXPERIMENTS.

By EWART S. ANDREWS, B.Sc. (Eng.).

The question of wind pressure is one that claims the closest attention of all interested in the design of buildings and in constructional engineering, and the article presented below should therefore be considered not only an interesting but an important contribution on the subject.—ED.

ALTHOUGH it is now four years since Dr. T. E. Stanton published the results of his experiments at the National Physical Laboratory on wind pressure, practical designers do not appear to have made any allowance in their calculations for the results of these experiments. In the present article we will outline the principal results of these experiments and will consider for one case the difference of the stresses in the members obtained, (*a*) in the usual manner, (*b*) on a method based upon Stanton's experiments.

PRELIMINARY EXPERIMENTS.

In 1903 Dr. Stanton carried out very careful experiments "On the resistance of plane surfaces in a uniform current of air."* A current of air was drawn by a motor-driven fan down a vertical channel 2 ft. in diameter, and plates and models were placed in this channel and the pressure upon them was measured by weighing apparatus. The velocity of the current of air was measured by means of Pitot tube apparatus and the intensity of the air-pressure was measured by means of a specially designed tilting water gauge. The principal result was the formula for the pressure on a plane normal to the wind

$$P = \cdot 0027 V^2$$

where P = resultant pressure in lbs. per sq. ft.

V = velocity of wind in miles per hour.

Dealing with models of roofs, it was found that there was an appreciable negative or suction pressure on the leeward side of the roof, and for roof-angles of 30° and 45° these negative pressures exceeded considerably the positive pressure on the windward side; in fact, for an angle of 30° the pressure on the windward side was practically nil. The results of these experiments with an artificial current of air gave promise of such important results that they were followed by experiments on a more ambitious scale with the wind itself.

LATER EXPERIMENTS.

These were carried out on a windmill tower erected on a fairly exposed position at Bushey Park, and the results were published in 1908.†

Mahogany boards of varying sizes from 10 ft. square were used, and the pressure was transmitted to a thin steel diaphragm communicating with a water gauge.

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* Proc. Inst. C.E., Vol. clvi.

† Proc. Inst. C.E., Vol. clxxi.

The general conclusion was that the *intensity of pressure is independent of the dimensions of the plate,* and is given by

$$P = \cdot 003\ V^2$$

where P and V are as before.

The roof models consisted of a steel principal, the roof angle of which was capable of adjustment from 30° to 60°; on the principal were carried mahogany boards 8 ft. by 7 ft.

The following results were obtained :—

Inclination (degrees)	Pressure per sq. ft. for $V = 20$ miles per hour	
	Windward	Leeward
60	+1·35	+·15
45	+1·13	0
30	+ ·61	−·16

The above figures apply to cases where roofs are supported on columns through which wind may pass.

Experiments were then made with a board at the side to obtain the effect of a wall on the leeward side, with the following results, which were obtained more than once :—

Inclination (degrees)	Pressure per sq. ft. on leeward side. $V = 20$ miles per hour
64	−·36
48	−·68
35	−·82

These results are of the same order as those on small models in a uniform current of air.

In order to find the effect of windows, doors, and like openings in the side of buildings, openings equal to 4 per cent. of the whole surface were made in the board at the side, with the following results :—

Inclination (degrees)	Conditions of Openings		Ratio of pressure inside to maximum pressure on windward side of a plate on which wind impinges normally
	Windward	Leeward	
60	—	Closed	1·00
60	—	Half Open	·67
60	—	Open	·20
—	Open	—	—
30	—	Closed	·82
30	—	Half Open	·49
30	—	Open	·20

Concluding results.—The following are given as the conclusions with regard to wind pressure on roofs, from which rules for design might be formulated.

Values of K for use in the formula:

$$P = KV$$

where P = pressure in lbs. per sq. ft.
V = velocity in miles per hour.

(*a*) Wind passing right through the columns.

	Values of K		
	60°	45°	30°
Windward Side ...	·0034	·0028	·0015
Leeward Side ...	nil	nil	nil

(*b*) Pressure possible inside building.

	Values of K		
	60°	45°	30°
Windward Side ...	+·0034	+·0028	+·0015
Leeward Side ...	−·0032	—	−·0022

APPLICATION TO DETERMINATION OF STRESSES.

We will now apply these results to the determination of the stresses in the roof truss shown in *Fig.* 1, and will compare the stresses obtained in the ordinary method with those obtained by the new method, allowing for suction pressure—*i.e.*, for the case where the wind does not blow straight through the columns.

The principals are taken as 10 ft. apart and the dead load as 18 lb. per sq. ft. of ground plan.

The stress diagram for this loading comes as shown in the figure, this diagram being drawn by the quite simple and well-known method of reciprocal figures. The loading shown is in thousands of pounds, or "*kips,*" as the American designers call it.

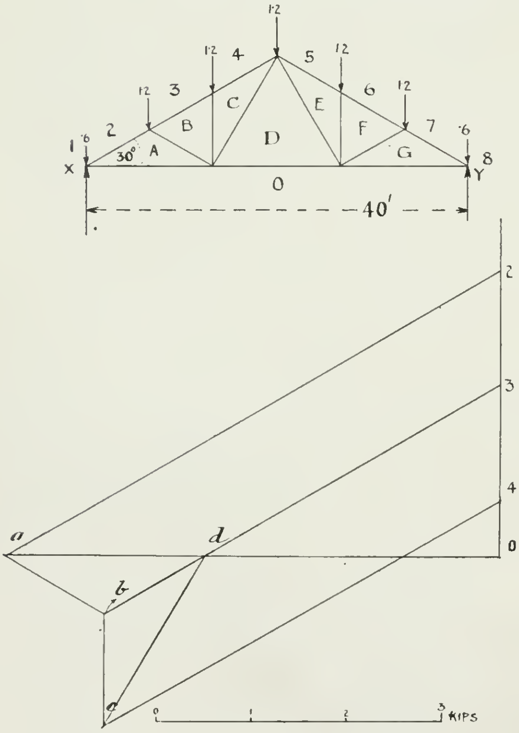


FIG. 1.

Fig. 2 shows the stress diagram for the wind pressure

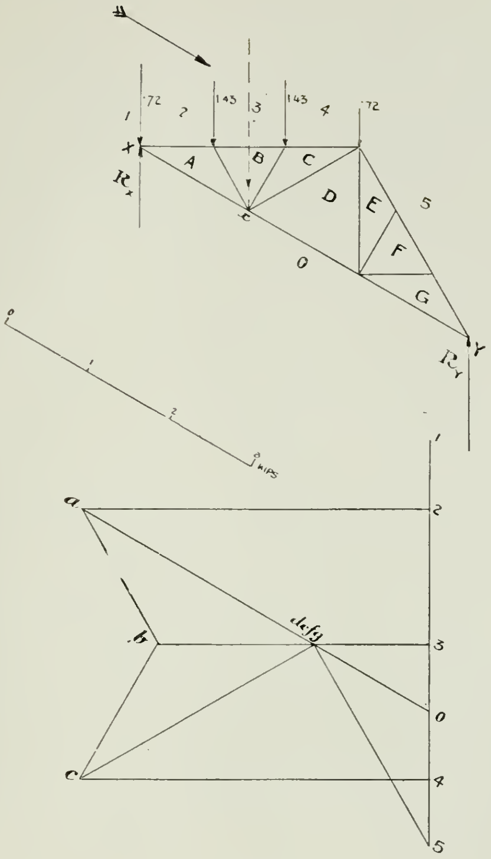


FIG 2.

drawn according to the usual manner of treating the wind; a wind pressure of 30 lb. per sq. ft. on a vertical surface is taken, which gives for a pitch of 30° a normal pressure of ·66 × 30 = 19·8 lb. per sq. ft., ·66 being the coefficient given by Hutton's formula, which is in most general use. The ends of the truss have been taken as fixed, so that the reaction at each end is parallel to the resultant wind pressure; there are arguments against this assumption, but those in its favour predominate, and it is accepted by most of the reliable writers.

To determine the amount of each reaction, and thus the point O on the reciprocal figure, we produce the total wind pressure to meet the span XY in the point x and divide the total pressure 1,5; so that

$$\frac{1,0}{0,5}=\frac{Xx}{xY}$$

The reciprocal figure is then drawn without difficulty and assumes a familiar form.

Now we come to the determination of the stresses in accordance with Stanton's experiments. According to the formula $P = ·003\ V^2$, 30 lb. per sq. ft. corresponds to a velocity of 100 miles per hour; taking, therefore, the figures from table (*b*) we have, for our case of 30° pitch—

Windward pressure = 15 lb. per sq. ft.
Leeward ,, = −22 lb. per sq. ft.

The wind loading on the truss then comes as shown in *Fig.* 3, the arrow, as in the previous figure, indicating the direction of the wind.

The resultant wind pressure acts at the intersection -Y- of the total windward and leeward pressures, which come to 15 × 23·09 × 10 = 3,462 and 22 × 23·09 × 10 = 5,080 lb. respectively.

The loads are divided up as shown (in kips, or thousands of pounds) and are set down to scale on the lines 1,4′ and 4′,8, the resultant wind pressure being given by 1,8; a parallel to 1,8 is drawn through the point y to meet XY, produced in x.

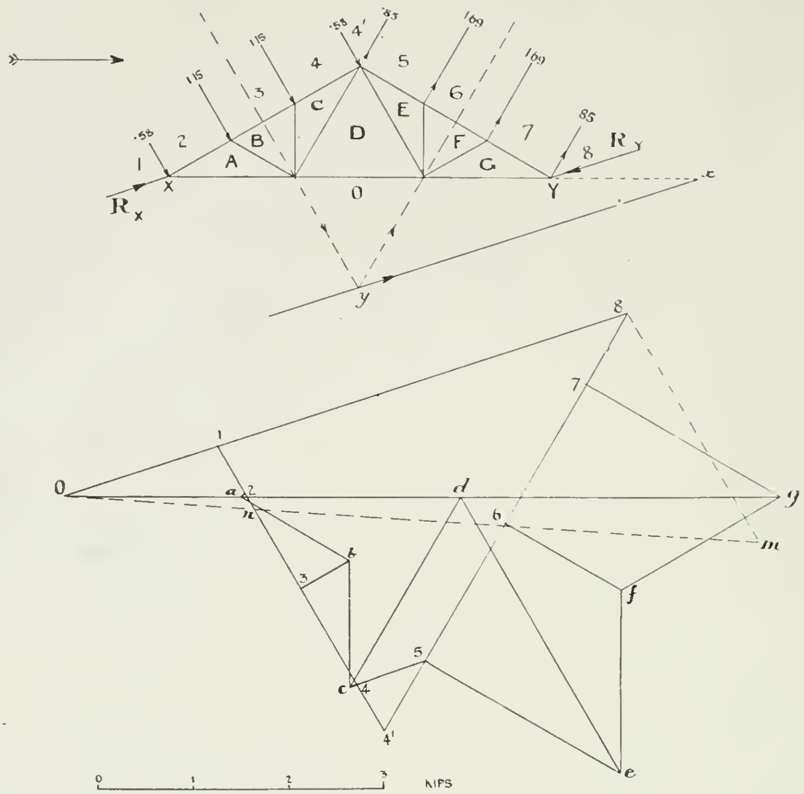


Fig. 3.

Then the point O is taken, so that

$$\frac{O1}{O8}=\frac{Xx}{Yx}$$

this may be done graphically by setting out $8m$ to represent Xx and $1n$ parallel to it to represent Yx on the same scale and joining across.

Then $8,O=Ry;\ O1=Rx.$

The reciprocal figure can then be drawn without much difficulty and comes to the form shown, which is very different from that shown in the previous figure.

We can then tabulate our results, and by noting that the stress in $G7$ say, when the wind is on the left will be the same as that in $2A$ when it is on the right, we get the stresses for the wind on each side.

STRESSES IN THOUSANDS OF POUNDS.

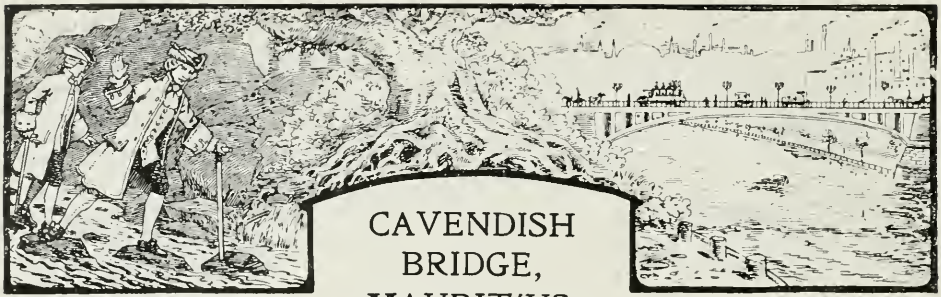
+ = compression. − = tension.

Member	Dead Load	Wind on ordinary method		Wind on new method		Maximum stress		Minimum stress	
		Wind Left	Wind Right	Wind Left	Wind Right	Old	New	Old	New
*A*2	+6·00	+3·67	+2·43	+0·06	−2·35	+9·67	+6·06	+6·00	+3·65
*B*3	+4·80	+2·86	+2·43	−0·58	−1·40	+7·66	+4·22	+4·80	+3·40
*C*4	+4·80	+3·67	+2·43	+0·06	−2·35	+8·47	+4·86	+4·80	+2·45
*A*0	−5·20	−4·24	−1·40	+1·84	+7·48	−9·44	−5·20 +2·28	−5·20	−3·36
*D*0	−3·12	−1·40	−1·40	+4·15	+4·15	−4·52	−3·12 +1·13	−3·12	+1·13
AB	+1·20	+1·64	nil	+1·30	−1·93	+2·84	+2·50	+1·20	−0·73
BC	+1·20	+1·64	nil	+1·30	−1·93	+2·84	+2·50	+1·20	−0·73
CD	−2·08	−2·86	nil	−2·30	+3·33	−4·94	−4·38 +1·25	−2·08	+1·25

Where reversal of stress occurs, two values are given in the maximum stress column.

GENERAL CONCLUSIONS.

An inspection of the above figures will show that the effect of the suction pressure is to reduce the maximum and minimum stresses in all the members, and to necessitate the lower members being made of sections such as angles or tees capable of taking compression; the latter point is often allowed for in practice. As many roof trusses are made of height equal to ¼-span it is rather a pity that the experiments were not extended to this slope, but it may be taken that results of the same order would be obtained, and leeward suction pressures in the same proportion to the windward pressures may be taken for design.



CAVENDISH BRIDGE, MAURITIUS.

By P. le JUGE de SEGRAIS, M.Inst.C.E.,
Director of Public Works, Mauritius.

It is interesting to note the increasing use of concrete and reinforced concrete in the Colonies, and below we present an article on a reinforced concrete bridge recently erected in Mauritius. We are indebted to the "Colonial Office Journal" for kindly placing the illustrations and matter at our disposal.—ED.

Previous to 1905 most of the road bridges of the Colony were built in timber, nineteen only out of 189 being constructed in masonry or steel. The sums spent annually for maintaining these timber bridges in good repair have amounted to about £4,000, and, with the object of doing away with this large recurrent expenditure, the policy of replacing gradually all the timber structures in material of a durable character has, on the suggestion of the author, been adopted by the Colonial Government.

As a result, three bridges have been entirely rebuilt in masonry (arches) and three with steel superstructures. Over thirty have been rebuilt in reinforced concrete, which has proved cheaper than either masonry or steel, and practically requires no maintenance. All the reinforced concrete bridges are periodically examined and show no sign of cracking anywhere.

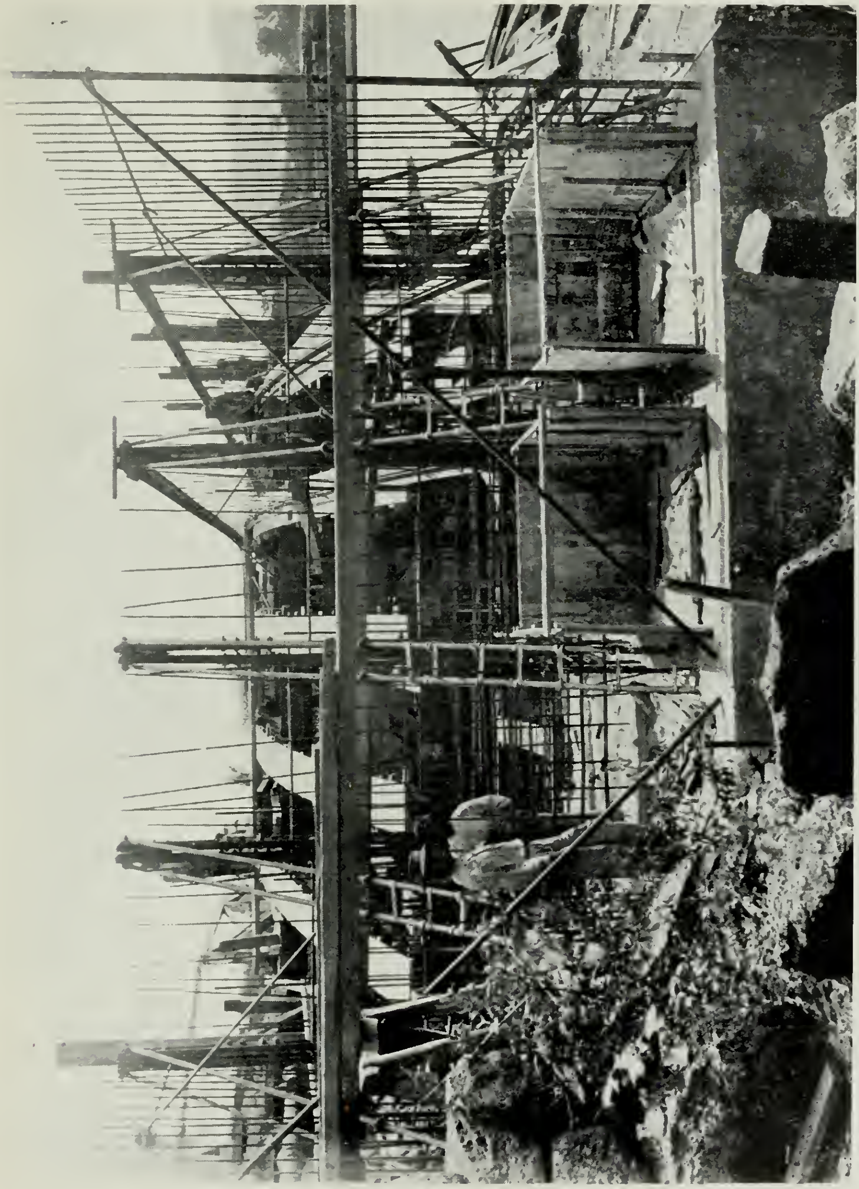
The existing abutments or piers of the timber bridges are generally of substantial basalt masonry, so that the superstructure only requires renewal, and this is done by blocking the road, if there is another road available within some reasonable distance, or the work is carried by halves, which is naturally not so satisfactory, and more costly also.

In some cases, however, it is necessary to rebuild the bridge entirely, either because there are no satisfactory abutments or piers, or because the position of the existing bridge is dangerous and difficult of approach, especially with the motor traffic, which is daily largely increasing.

The bridge just completed and called Cavendish Bridge is the longest road bridge in the Island.

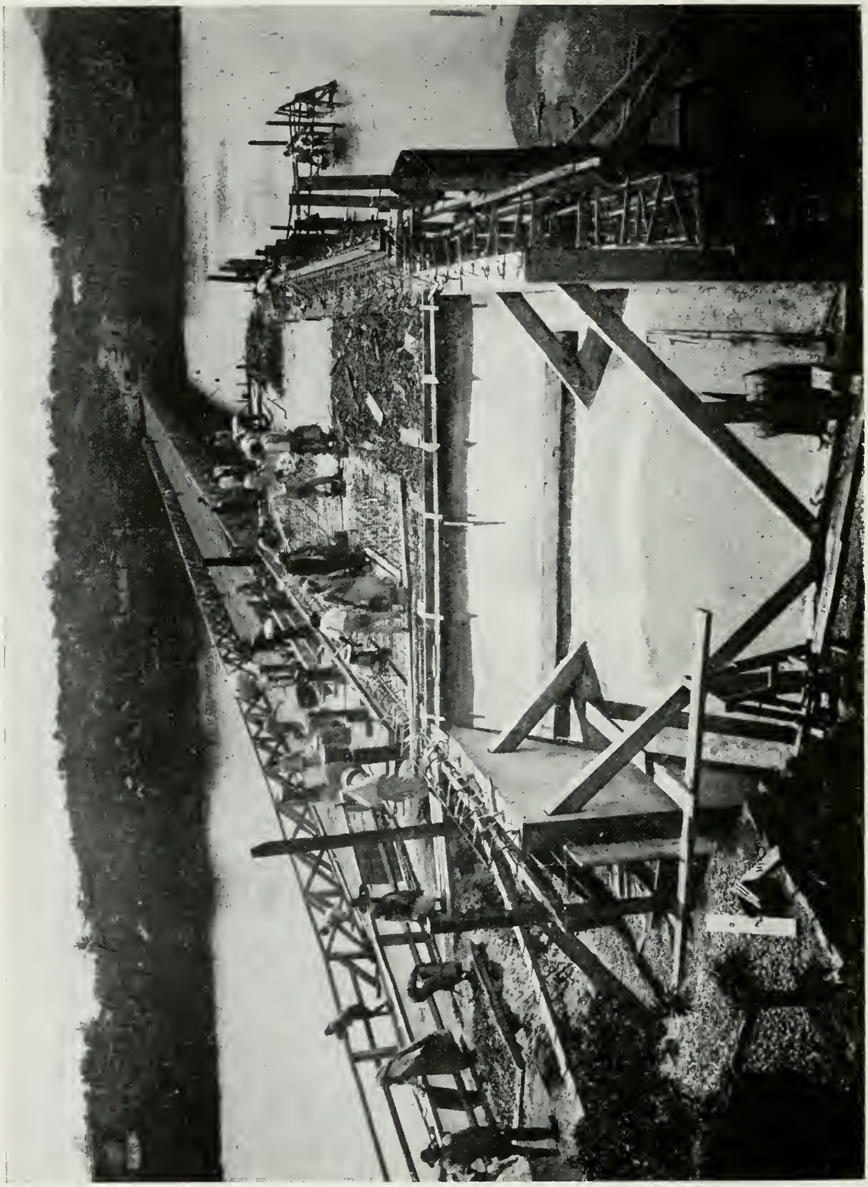
The old structure, built about the year 1856, consisted of 35 pairs of timber piles, spaced 18 to 20 ft. apart, across the estuary of River "La Chaux," to connect the town of Mahebourg with its suburb called "Ville Noire," and establish communication between Mahebourg and an important section of the district of Grand Port. Only light traffic, such as carts and carriages, was permissible over this bridge, and now and then a roller, not exceeding 5 tons in weight, was allowed to be dragged over the bridge by men with the greatest precautions.

The cross section of this old timber bridge shows how the work was carried out, the timber piles, 10 to 12 in. square, driven about 20 ft. in unsafe ground, being connected top and bottom by horizontal timber ties and struts, on which the timber platform, 4 in. thick, was laid across longitudinals fixed in the top horizontal member. It will be understood that a structure of this description, measuring 533 ft. from end to end (there being no masonry abutments, but piles driven at each end were secured in dry walling to form the abutments),



View of Southern Abutment.
THE CAVENDISH BRIDGE, MAURITIUS.

was not very satisfactory as regards stability; in fact, it was so shaky that it was with difficulty that the author could use his level while standing on the bridge, the spirit bulb being thrown out of level by any person passing the instrument.



View of Northern Abutment.
THE CAVENDISH BRIDGE, MAURITIUS.

The new bridge was designed with a view to facilitate the traffic, and to carry a moving load of one ton per foot run, which is more than ample for the local requirements.

A good view of the bridge is given by the photograph taken on the down stream side (*See Frontis.*). There are seven flat arches, span 62 ft. 6 in. and rise 1-10 of the span, or 6 ft. 3 in., the piers at the springings of the arches being 5 ft. thick, and the abutments 20 ft. deep, giving a total length of 507 ft. 6 in. for the portion of the work done in reinforced concrete. The approaches, measuring 82 ft. and 177 ft. respectively, are made in coursed rubble masonry with a finely dressed ashlar parapet.

Abutments.—The southern abutment on the Mahebourg side rests on 20 teak piles, 10 in. square, driven in the solid 10 to 20 ft.; these piles support a reinforced concrete base, 2 ft. thick, by means of $1\frac{1}{2}$-in. round steel bars fixed on the top of the piles, to which a lattice work of $\frac{5}{8}$-in. and $\frac{1}{2}$-in. round steels fixed 6 in. apart are connected. Light walls of reinforced concrete were then built 1 ft. thick, leaving hollows which were filled up with coral sand.

The abutment has the shape of a trapezium, with the exception of the two outside walls, which are tied by reinforced beams into the central wall. This is shown by the photographs of the abutment on pages 675 and 676. The light walls are reinforced by vertical $\frac{5}{8}$-in. round steel bars placed 8 in. apart, and horizontal $\frac{1}{2}$-in. round steel bars similarly spaced.

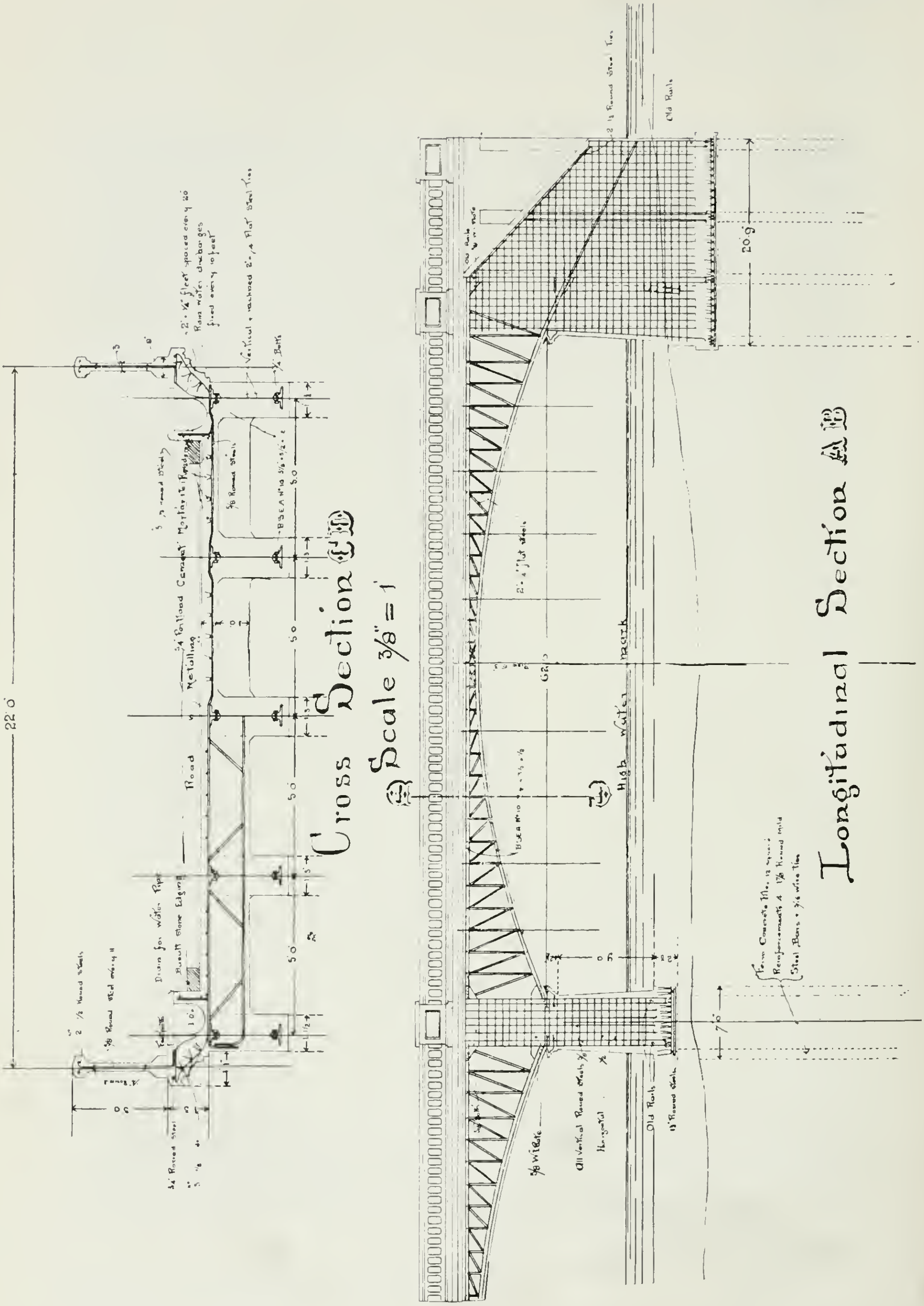
The walls are further reinforced by five pairs of unserviceable rails where the arch reinforcements are attached, and also a second line of ten old rails at the back which serve for further anchoring the arch reinforcements by means of two pairs of $1\frac{1}{2}$-in. round steel bars.

Some difficulty was experienced in making the foundations. It was first of all anticipated that it would be possible to find solid ground from trial pits previously made, but water percolating made the excavation very difficult, especially at high tide, and steam pumping had to be resorted to. It was then found preferable to drive piles to support the concrete.

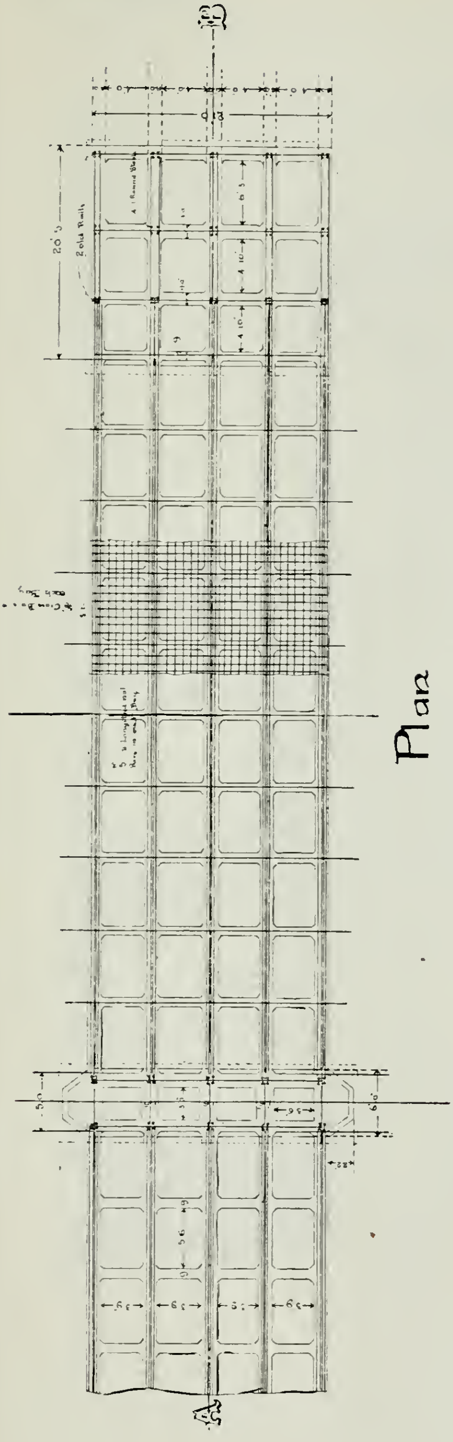
The northern abutment in "Ville Noire" was built in the river bed, as well as a part of the masonry approach wall, and a cofferdam was first of all made; fifteen reinforced concrete piles were driven to support the abutment, which was, in other respects, the reproduction of the southern one. Below water these piles were connected together in the same manner as those for the piers, as explained further on. Sixteen reinforced concrete piles were subsequently driven to support the masonry walls of the approach on that side, the piles being spaced 18 ft. and 4 ft. centres, and connected by a reinforced concrete base 6 ft. wide and 1 ft. thick, on which the masonry walls, 17 ft. high at the deepest part on each side, were built without difficulty.

Each pile was reinforced with four $1\frac{7}{8}$-in. round mild steel bars, tied with $\frac{3}{16}$-in. wire spaced close at the point and end, and nowhere less than 6 in. apart. The piles were moulded horizontally always two months in advance; they were driven by a $1\frac{1}{2}$-ton cast iron monkey with a drop not exceeding 7 ft. until the piles reached the firm ground, that is when the penetration per blow did not exceed $\frac{1}{8}$ in. No difficulty was experienced in driving the piles with the interposition of a wooden dolly and a steel square box fitted on top of the pile and filled with sawdust and wooden shavings.

Piers.—A cofferdam had to be made for each pier. Ten reinforced concrete piles were driven from 15 ft. to 24 ft. In one case the piles had to be lengthened, and in others they had to be cut off.



Sections.
The Cavendish Bridge, Mauritius.



The Cavendish Bridge, Mauritius.

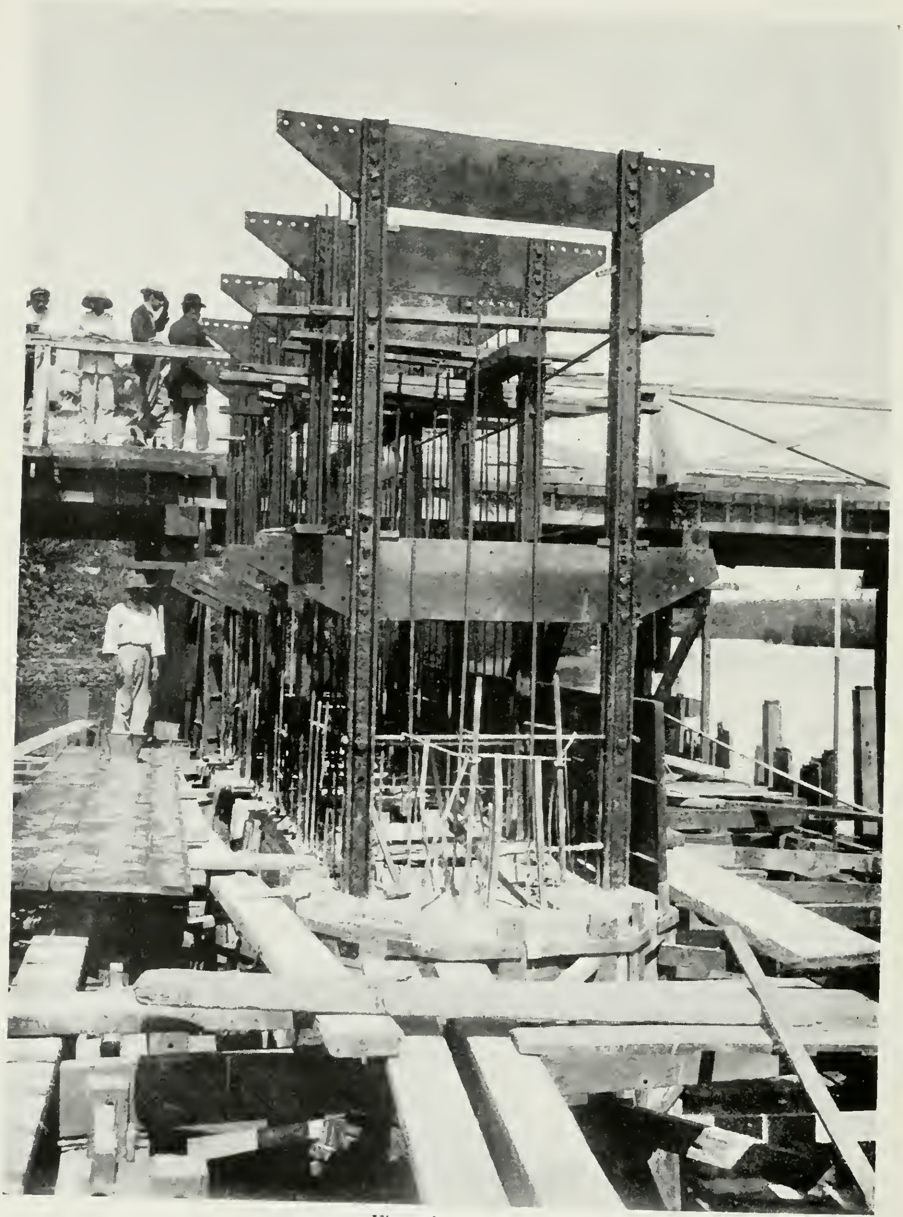
These piles were connected by short reinforced concrete walls, 1 ft. thick and 2 ft. to 3 ft. deep—*i.e.*, about 5 ft. below low water level—to protect the piles further down in case of erosion. The reinforced concrete base, 26 ft. long, 7 ft. wide, and 27 in. thick, resting on the piles, is reinforced by two $1\frac{1}{2}$-in. steel bars resting on the tops of the piles lengthwise and crosswise, the intervals being fitted up with a netting of $\frac{5}{8}$-in. and $\frac{1}{2}$-in. round steels, spaced 6 in. apart. When a foot of concrete had been laid the old rails were fixed in proper alignment, these being fixed in pairs, 5 ft. centres, and the remaining 15 in. of concrete were then laid. Each pier has four hollows 4 ft. by 3 ft. 6 in.; these were left empty to save weight on the piles and give more elasticity to the pier. The walls were reinforced in the same way as those of the abutment.

Arches.—Each span consists of five beams, the three central ones 15 in. thick and the outside ones $13\frac{1}{2}$ in.; the depth of each beam is 1 ft. at the centre and 7 ft. 3 in. at the haunches.

The concrete platform is tied on the top of these beams, which are connected crosswise by nine cross beams 9 in. thick and 12 in. deep. These beams form continuous beams from one end of the bridge to the other, and are firmly tied into the abutments, as already explained. The reinforcement consists of two $3\frac{1}{2}$-in. by $3\frac{1}{2}$-in. by $\frac{1}{2}$-in. mild steel angles forming two inverted tees top and bottom, which are connected by vertical and diagonal bracings bolted to the steel angles by $\frac{3}{4}$-in. bolts spaced 2 ft. 1 in. apart.

The lower angles are placed 2 in. below the intrados, and the upper ones project 3 in. into the 6-in. platform.

The cross beams are reinforced by 5 1-in. round mild steel bars, one of each passes over the upper steel angles and is inclined towards the centre of each bay to provide against shearing. The platform consists of a series of ⅝-in. and ½-in. round mild steel bars forming a square of 8 in. with stirrups (made of cement cask steel bands) at each intersection.



View of Pier Construction.
The Cavendish Bridge, Mauritius

The concreting of each beam took one day, and that of the platform three days on each occasion; a rendering of ¾ in. of cement mortar 1 to 1 was laid immediately after, and the whole kept moist for a fortnight. The cornice on which the handrail is built was next concreted.

Each span was made in succession, and it is to be noticed that there is no crack noticeable where new work was laid against old on six different occasions at intervals of four to six months.

The composition of the concrete used was a mixture of 12 parts by volume of carefully cleaned and washed basalt macadam of ¾ in. and 1½ in. in size, 6 parts of screened coral sand, and 2 parts of Portland cement for all the walls in compression, and 3 parts of cement for the superstructures.

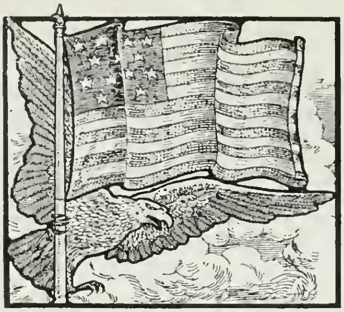
All the outside work was carefully rendered and polished with fine cement mortar 1 to 1, and by fixing small wedge-shaped battens inside the moulds to imitate masonry joints the work has the appearance of being made of blocks of marble.

It took a little over three years to build the bridge. The work was started on February 17th, 1908, and the bridge tested on March 28th, 1911, when, on passing a traction engine weighing 17 tons and three trailers loaded with bags of sugar each weighing 9 tons, a maximum deflection of 1·24 in. was obtained.

The cost of the work is given below. The old timber bridge was built originally for a contract price of $47,000, equivalent to £9,400; the timber was naturally periodically renewed. The sums spent for repairs to the bridge for a period of nine years, from 1896 to 1904, amounted to Rs. 36,425, giving an average sum of Rs. 4,000 in round figures, or £266, for annual maintenance.

The following is the detailed schedule of cost :—

	£	s.	d.	£	s.	d.
1. *Reinforced Concrete Work—*						
Portland cement from England	1,296	4	7			
Mild steel from England	1,272	3	10			
Timber for false work	248	12	4			
Labour	3,456	18	8			
Sand	63	5	4			
Cartage, dressed stones for caps of parapets, edge of roadway, incidentals	380	18	7			
				6,718	3	4
2. *Masonry Approaches—*						
Portland cement from England	281	11	9			
Steel from England	82	0	11			
Labour	1,098	0	3			
Sand	11	11	10			
Cartage, dressed stones, and incidentals	445	10	4			
				1,918	15	1
Total				£8,636	18	5



PROPOSED STANDARD SPECIFICATIONS FOR REINFORCED CONCRETE WORK.

PRESENTED AT THE CONVENTION OF THE NATIONAL ASSOCIATION OF CEMENT USERS.

In our August number we printed two of the Papers presented at the last Annual Convention of the National Cement Users' Association. We now publish some of the proposed standard specifications for reinforced concrete floors, which should be of particular interest to our readers.—Ed.

PROPOSED STANDARD SPECIFICATIONS FOR REINFORCED CONCRETE FLOORS.

Materials.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials and adopted by this Association. (Standard No. 1.)

2. *Aggregates.*—The aggregates shall be clean, coarse, hard, durable materials, and shall be free from dust, soft, flat or elongated particles, loam, vegetable or other deleterious matter. In no case shall aggregate containing frost or lumps of frozen material be used.

(*a*) *Fine Aggregate for Concrete.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, preferably of silicious materal, graded from fine to coarse, and passing, when dry, a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and not more than three (3) per cent. shall pass a sieve having one hundred (100) meshes per linear inch.

Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate, by weight, when made into briquettes will show a tensile strength at least equal to the strength of 1 : 3 mortar of the same consistency made with the same cement and Standard Ottawa sand.

(*b*) *Aggregate for Wearing Course.*—The aggregate shall consist of screened gravel or stone screenings from granite or other close-grained durable rock, sufficiently hard to scratch glass, mixed in the proportion of three (3) parts passing a one-half ($\frac{1}{2}$) inch ring and retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and two (2) parts passing a screen having one-quarter ($\frac{1}{4}$) inch diameter holes and retained on a screen having fifty (50) meshes per linear inch.

(*c*) *Coarse Aggregate for Concrete.*—Coarse aggregate shall consist of inert materials, such as crushed stone or gravel, graded in size, retained on a screen having one-quarter ($\frac{1}{4}$) inch diameter holes, and the maximum size shall be such as to pass a one and one-quarter ($1\frac{1}{4}$) inch ring.

(*d*) *Natural Mixed Aggregates.*—Natural mixed aggregates shall not be used as they come from the deposit, but shall be screened and remixed to agree with the proportions specified.

3. Water shall be clean, free from oil, acid, alkali, or vegetable matter.

4. If artificial colouring material is required, only mineral colours shall be used.

5. The reinforcement shall meet the requirements of the Standard Specifications for Steel Reinforcement adopted March 16th, 1910, by the American Railway Engineering Association.

Construction.

6. *Forms.*—The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed in less than ten (10) days after the concrete is placed, and then only with the consent of the engineer in charge.

7. *Reinforcement.*—The reinforcement shall be free from rust, scale or coatings of any character which will tend to reduce or destroy the bond, and shall be placed and held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at the point of maximum stress shall be avoided.

Measuring and Mixing.

8. *Measuring.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate, uniform proportions at all times. A bag of Portland cement (94lb. net) shall be considered one (1) cubic foot.

9. *Mixing.*—The ingredients of the concrete or mortar shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency and the mixing continued until the materials are uniformly distributed and the mass is uniform in colour and homogeneous.

(*a*) *Machine Mixing.*—When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass, shall be used.

(*b*) *Hand Mixing.*—When it is necessary to mix by hand, the mixing shall be on a water-tight platform and the materials shall be turned until the mass is uniform in colour and homogenous.

10. *Retempering*—that is, remixing mortar or concrete that has partially hardened with additional water—will not be permitted.

Two-Course Floors.

11. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two (2) parts fine aggregate, and four (4) parts coarse aggregate.

12. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flow into the forms and about the reinforcement, but which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar

13. *Placing.*—When it is necessary to make a joint in a floor slab, its location shall be designated by the engineer. The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. It shall be brought to a surface at least one (1) inch below the finished surface of the floor and as soon as the slab has hardened sufficiently to permit of workmen being upon it, and before it has dried out, the wearing course shall be applied. Workmen shall not be permitted to walk on freshly laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

14. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion of one (1) part Portland cement and not more than two (2) parts aggregate for wearing course.

15. *Consistency.*—The mortar shall be of a consistency that water will flush to the surface only under heavy, thorough working with a wood float.

16. *Thickness.*—The wearing course of the floor shall have a minimum thickness of one (1) inch.

17. *Placing.*—The wearing course shall be placed immediately after mixing.

18. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner which will thoroughly compact it. When required, the surface shall be trowelled smooth, but excessive working with a steel trowel shall be avoided. If excessive moisture occurs on the surface, it must be taken up with a rag or mop, and in no case shall dry cement or a mixture of dry cement and sand be used to absorb this moisture or to hasten the hardening.

19. *Colouring.*—If artificial colouring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform colour. In no case shall the amount of colouring exceed five (5) per cent. of the weight of the cement.

Wearing Surface on Hardened Slabs.

The general requirements of the specifications covering two course work will apply to Wearing Surface on Hardened Slabs with the following exceptions:

20. All dirt and debris shall be removed from the surface of the slab and the surface roughened in a manner that will expose the freshly broken face of the coarse aggregate. The floor shall be thoroughly scrubbed with a warm soap, lye, or acid solution, and rinsed with clear water, preferably under pressure. All water used to clean the slab shall be taken up with a rag or mop or otherwise disposed of.

21. *Treatment of Slab.*—The slab shall be thoroughly wet at the time the wearing course is placed. The surface of the slab shall be carefully treated with a neat cement grout brushed on, or with such a preparation as the engineer may direct.

22. *Consistency.*—The mortar for the lower one-quarter ($\frac{1}{4}$) inch shall be of such a consistency that it can be easily spread with a straight edge. The mortar for the remainder of the wearing surface shall be of the consistency specified for wearing surface.

23. *Placing.*—The wearing surface shall be placed in two operations. Upon the freshly placed cement grout, a one-quarter ($\frac{1}{4}$) inch layer of mortar of the consistency specified shall be placed and thoroughly worked into the irregularities of the surface of the slab, and immediately followed with sufficient mortar of the consistency specified for wearing course to bring the floor to the established grade.

Protection.

24. *Treatment.*—When completed, the floor shall be covered with at least two (2) inches of wet sand or sawdust which shall be kept moist for four (4) days, and no one shall be allowed on the floor during that time.

25. *Temperature below 35° F.*—If at any time during the progress of the work the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to 35 degrees Fahrenheit, the water and aggregate shall be heated and precautions taken to protect the work from freezing for at least five days. In no case shall concrete be deposited upon a frozen base.

PROPOSED REVISED STANDARD SPECIFICATIONS FOR CONCRETE ROADS AND STREET PAVEMENTS.

Materials.

PARAGRAPHS 1 to 6, giving specifications of cement and aggregates, are the the same as in the specification for reinforced concrete floors (page 682).

7. *Joint Filler.*—The expansion joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather.

Sub-Grade.*

8. *Section.*—The sub-grade shall have a rise at the centre of not more than one-hundredth (1/100) the width of the pavement.

9. *Depth.**—(*a*) The sub-grade shall not be less than twelve (12) inches below the finished surface of the pavement.

(*b*) The sub-grade shall not be less than six (6) inches below the finished surface of the pavement.

10. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) inches in thickness.

11. *Deep Fills.*—When a fill exceeding one (1) foot in thickness is required to bring the pavement to grade, it shall be made in a manner satisfactory to the engineer.

12. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

Sub-Base.

13. *Thickness.*—On the sub-grade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled and tamped to a surface at least six (6) inches below the finished grade of the pavement.

14. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet and shall be in that condition when the concrete is deposited.

Forms.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the pavement.

17. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

Expansion Joints.

18. *Width and Location.*—One-half ($\frac{1}{2}$) inch expansion joints shall be placed across the street or road not more than twenty-five (25) feet apart, perpendicular to the centre line. When a curb or combination curb and gutter is used, a similar joint shall be placed betweeen it and the pavement. All expansion joints shall extend through the entire thickness of the pavement.

19. *Protection of Edges.*—Where required by the engineer in charge, the expansion joints shall be protected with metal. Unless protected by metal, the upper edges of the concrete shall be rounded to a radius of one-half ($\frac{1}{2}$) inch.

Measuring and Mixing.

The same requirements are laid down as in the specification for reinforced concrete floors (page 683).

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* NOTE.—When a sub-base is required, eliminate paragraph 9 (*b*). When a sub-base is not required, eliminate paragraphs 3 and 9 (*a*). Unless paragraph 9 (*a*) is eliminated, 9 (*b*) is void.

Two-Course Pavement—Base.

23. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement, two and one-half ($2\frac{1}{2}$) parts fine aggregate, and five (5) parts coarse aggregate.

24. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

25. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections. Under no circumstances shall concrete be used that has partially hardened. The concrete shall be well tamped to a surface the thickness of the wearing course below the established grade of the pavement. Workmen shall not be permitted to walk on the freshly laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

26. On streets more than twenty-five (25) feet wide not having car tracks, the pavement shall be reinforced with wire fabric or with plain or deformed bars. The cross sectional area of metal shall amount to at least 0·041 sq. in. per foot measured parallel to the axis of the street and at least 0·025 sq. in. per foot measured perpendicular to the axis of the street. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. Reinforcement shall not cross expansion joints and shall be lapped sufficiently to develop the strength of the metal.

Wearing Course.

27. *Proportions.*—The mortar shall be mixed in the manner hereinbefore specified in the proportion by volume of one (1) part Portland cement and not more than two (2) parts aggregate for wearing course.

28. *Consistency.*—The mortar shall be of a consistency that will not require tamping, but which can be easily spread into position.

29. *Thickness.*—The wearing course of the pavement in residence districts shall have a minimum thickness of one and one-half ($1\frac{1}{2}$) inches and in business districts a minimum of two (2) inches.

30. *Placing.*—The wearing course shall be placed immediately after mixing, and in no case shall more than fifty (50) minutes elapse between the time the concrete for the base is mixed and the time the wearing course is placed.

31. *Finishing.*—After the wearing course has been brought to the established grade, it shall be worked with a wood float in a manner that will thoroughly compact it. Before the wearing course has completely hardened, it shall be roughened by brushing with a stiff vegetable fibre brush or broom. On grades of over five per cent. (5 %), the surface shall be corrugated if directed by the engineer.

32. If artificial colouring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform colour. In no case shall the amount of colouring used exceed five (5) per cent. of the weight of the cement.

One-Course Pavement.

The general requirements of the specifications covering two course work will apply to one course work with the following exceptions :—

33. *Proportions.*—The concrete shall be mixed in the proportion by volume of one (1) part Portland cement to one and one-half ($1\frac{1}{2}$) parts fine aggregate (Paragraph *2a*) or aggregate for wearing course (Paragraph *2b*) and three (3) parts coarse aggregate passing a one (1) inch ring.

34. *Placing and Finishing.*—The concrete shall be placed as provided for under Two-Course Pavement, tamped and struck off to grade. The coarse particles shall

then be forced to a depth below the surface which will permit of finishing as specified under Two-Course Pavement.

35. *Reinforcement.*—When a one-course pavement is to be reinforced, the metal shall be placed two (2) inches from the upper surface of the pavement. The minimum amount of metal shall be as specified under Two Course Pavement.

Wearing Surface of Bitumen and Sand.

36. *Proportions.*—When a wearing surface of bitumen and sand is used, it shall be placed upon a one-course pavement constructed as hereinbefore specified.

37. *Bitumen.*—The bitumen shall be of a quality specified by the engineer.

38. *Placing.*—After the concrete base has hardened for at least seven (7) days, the clean, dry surface of the pavement shall be covered with hot bitumen applied with a sprinkling wagon designed for the purpose, or with suitable hand sprinkling cans. The hot bitumen shall be evenly distributed over the concrete by brushing with suitable brooms immediately after applying, and then covered with the fine aggregate. (Paragraph 2*b*.)

39. *Amount of Bitumen and Sand.*—Between one-third ($\frac{1}{3}$) and one-half ($\frac{1}{2}$) gallon of bitumen shall be applied per square yard of pavement. Approximately one (1) cubic yard of fine aggregate shall be applied per one hundred and fifty (150) square yards of pavement.

40. *Expansion Joints.*—Before applying the wearing surface of bitumen and sand, all expansion joints in the pavement shall be filled as hereinbefore specified. Where a wearing surface of bitumen and sand is used, the edge of the expansion joint need not be protected with metal.

Crown.

41. *Amount.*—All types of concrete pavement shall be given a rise or crown at the centre of at least one one-hundredth (1/100) but of not more than one seventy-fifth (1/75) of the width of the pavement. A portion of this crown may be obtained by increasing the thickness of the pavement at the centre rather than by laying a pavement of uniform thickness on a crowned sub-grade or sub-base.

Protection.

42. *Treatment.*—When completed, the work shall be kept moist and protected from the elements for at least three (3) days, and the pavement shall not be open to traffic until the engineer so directs.

43. *Temperature below 35° F.*—If at any time during the progress of the work the temperature is, or in the opinion of the engineer will, within twenty-four hours, drop to thirty-five (35) degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

PROPOSED RECOMMENDED PRACTICE FOR PLAIN CONCRETE DRAIN TILE.

Materials.

The same requirements as laid down for the cement and aggregates as in the specification for reinforced concrete floors (page 682).

Proportions.

The proportions of cement to aggregate shall be such as to require at least the minimum amount of cement to produce strength and density as hereinafter specified. The proportions of the various sizes of aggregates and cement to aggregates shall preferably be made by weight. If by volume, a bag of Portland cement shall be considered 1 cubic foot.

Methods of measurement of the proportions of the various ingredients shall be used which will secure uniform measurements at all times.

Mixing.

The ingredients of concrete shall be thoroughly mixed dry, sufficient water added to obtain the desired consistency, and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in colour and homogeneous. The mixing shall preferably be done with a machine mixer of a type which insures the proper mixing of the materials throughout the mass.

(*a*) *Semi-wet Process.*—The consistency of the concrete shall be such as to show web-like markings on the surface of the tile when the forms are removed.

(*b*) *Wet Process.*—The consistency of the concrete shall be such that it will be forced into every part of the mould by jarring or tapping.

Retempering or using mortar or concrete 40 minutes after being mixed shall not be permitted.

Curing.

Natural Curing.—The tile shall be protected from the sun and strong currents of air for a period of at least seven days. During this period they shall be sprinkled at such intervals as is necessary to prevent drying, and maintained at a temperature of not less than 50° F. Such other precautions shall be taken as to enable the hardening to take place under the most favourable conditions. After seven days, the tile may be removed to the yard, but in no case used before they are 28 days old.

Steam Curing.—The tile shall be removed from the moulds as soon as conditions will permit and shall be placed in an atmosphere of steam saturated with moisture for a period of at least forty-eight hours. The tile shall then be removed and stored for at least eight days. The steam-curing chamber shall contain an atmosphere saturated with moisture and maintained at a temperature between 100° and 130° F. (This does not apply to pressure steam curing.)

Dimensions.

(*a*) *Diameter.*—The diameter or size of the tile shall refer to the inside diameter and be uniform in all directions.

(*b*) *Thickness.*—From 4 to and including 22 in. in diameter, the wall thickness shall not be less than one-twelfth of the diameter. Tile above 22in. in diameter shall have a wall thickness of not less than one-tenth of the diameter.

(*c*) *Length.*—The length of the tile shall be uniform at all points, and preferably not less than the diameter, with a minimum of 12 in.

PROPOSED STANDARD SPECIFICATIONS FOR CONCRETE ARCHITECTURAL STONE, BUILDING BLOCKS AND BRICK.

I. Test Requirements.

1. Concrete architectural stone, building blocks and brick must be subjected to compression and absorption tests. The test samples must represent the ordinary commercial product of the regular size and shape used in construction.

2. *Compression.*—(*a*) *Solid concrete, stone, block, and brick.*—In the case of solid concrete, stone, block, and brick the ultimate compressive strength at 28 days must average 1,500 lb. per sq. in. of gross cross-sectional area of the stone as used in the wall, and must not fall below 1,050 lb. per sq. in. in any case.

(*b*) *Hollow and two-piece building blocks.*—The ultimate compressive strength of hollow and two-piece building blocks at 28 days must average 1,000 lb. per sq. in. of gross cross-sectional area of the block as used in the wall, and must not fall below 700 lb. per sq. in. in any case.

(*c*) *Area of hollow blocks.*—In the case of hollow building blocks the gross cross-sectional area shall be considered as the actual wall area, including the block and air space displaced by the block.

(*d*) *Area of two-piece blocks.*—In the case of two-piece blocks the blocks shall be tested in pairs consisting of the front and rear blocks as used in the wall. The compressive strength shall be regarded as the sum total sustained by the two blocks divided by the product of the length of the blocks and the width of the wall.

3. *Absorption.*—The percentage of absorption at 28 days (being the weight of the water absorbed, divided by the weight of the dry sample) must not exceed five (5) per cent. when tested, as hereinafter specified.

II.—Standard Methods of Testing.

4. *General.*—(*a*) *Laboratory.*—All tests required for approval shall be made in some laboratory of recognised standing.

(*b*) *Samples.*—For the purpose of the tests at least nine samples or test pieces must be provided. Such samples must represent the ordinary commercial product, and shall be selected from stock. In cases where the material is made and used in special shapes or forms too large for testing in the ordinary machines, smaller size specimens shall be used as may be directed.

(*c*) *Tests.*—Tests shall be made in a series of at least three. The remaining samples are kept in reserve in case duplicate or confirmatory tests be required. All samples must be marked for identification and comparison.

5. *Compression Tests.*—The compression tests shall be made as follows:

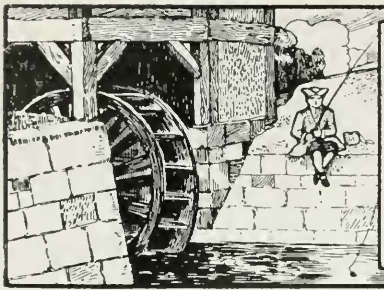
(*a*) *Solid concrete stone, block, and brick.*—When testing solid concrete stone, block, and brick, the net area shall be considered as the minimum area in compression.

(*b*) *Hollow and two-piece building blocks.*—Whenever possible such tests shall be made on full sized blocks. When such tests must be made on portions of blocks, both pieces of the block must be tested and both must contain at least one full web section. The samples must be carefully measured, then bedded flatwise in plaster of Paris to secure uniform bearing in the testing machine and crushed.

The net area shall be regarded as the smallest bearing area of the piece being tested. The total compressive strength shall be divided by the net area to obtain the compressive strength in lb. per sq. in. of net area of each piece. The sum of the two results shall then be averaged to obtain the average strength in lb. per sq. in. of the net area of the total block.

The entire block shall be carefully measured to determine the maximum air space prior to breaking the block for the compressing tests, and the net compressive strength obtained shall then be reduced to compressive strength in lb. per sq. in. of gross area, this being figured from the actual air space of the block determined above.

6. *Absorption Tests.*—The sample is first thoroughly dried to a constant weight at not to exceed 212° F. and the weight carefully recorded. When dried, the sample is to be immersed in a pan or tray of water to a depth of 2 in. resting on two strips not over 1 in. in width to allow the water to have free access to face. At the end of 48 hours from the time it is placed in water, the sample shall be removed, the surface water wiped off, and the sample carefully weighed.



USE AND ABUSE OF CONCRETE MACHINERY.

With the increasing use of machinery on works where concrete is being employed, it behoves contractors to be more careful in the selection of engine drivers and mechanics than heretofore, if the full efficiency of modern plant is to be obtained. Repairs require intelligent handling. The following instance of faults found in concrete machinery, on actual jobs, is taken from "The Cement Age" of U.S.A.—ED.

IN a pulverising machine one of the main shafts was unable to perform its mission due to the wearing away of the broken cap of a box. *Fig.* 1 shows how the box was fixed. A piece of sheet metal ¼ in. was procured and shaped to fit over the form of the box cap. Holes were drilled through the metal and the cast-iron cap and rivets were headed up in these holes, thus holding the part together quite securely. One of the adjusting set screws of the cap was utilised to assist in holding down the wrought-iron form on the cap. Apparently this was a suitable job, but the defective cap metal was forced irregularly down upon the shaft. This resulted in heating and binding. The machine ran hard. The workmen complained. Finally, a new complete box and cap were substituted, after which no more trouble was experienced. This case is referred to because it often happens that a very high standard concrete-working machine is ill-treated in this manner, and the machine gets the blame for non-effective service.

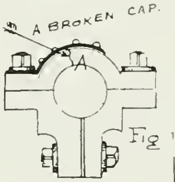


Fig 1

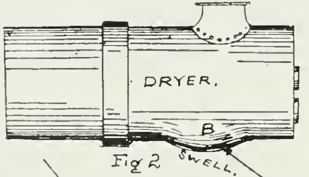


Fig 2

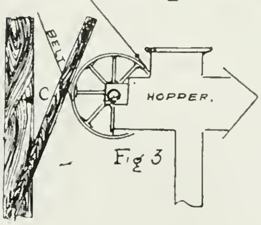


Fig 3

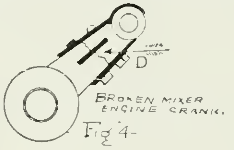


Fig 4

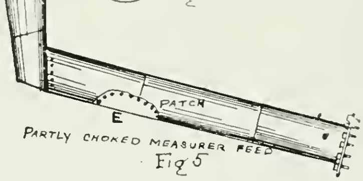


Fig 5

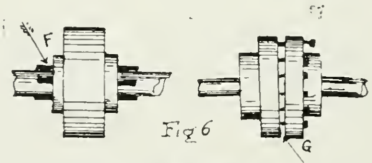


Fig 6

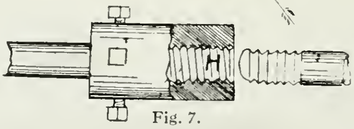


Fig. 7.

In a dryer which was in use in a certain concrete works the boiler plate was bulged as at B, *Fig.* 2. An examination revealed that this swelling was not due to a defective plate in the boiler, but to the fact that the attendant had permitted scale to gather inside. This scale prevented the water from properly contacting with the plate. Hence the cooling influence was gone. The intensity of the heat striking continuously upon the plate made the plate bulge. In order to remedy this defect it was necessary to cut out the bulged plate. Then a new plate was riveted in. To prevent repetition of this kind of swelling of the plate, directions were given to clean the scale from the interior of the boiler often enough to insure effective removal of the corrosive matter.

A hopper, *Fig.* 3, was giving poor work in a plant. It was found that the hopper itself was all right, although the workman said that there was something wrong with it. The workman had nailed up a piece of stick, C, in order to make the belt keep in place on the wheel. This is a most defective way of making a belt keep its course. The stick had to be removed, and by correct alignment of the shafting the belt made to run true on the wheels. Furthermore, instead of nailing up a wooden stick for a guide, a regular prong shipper was put on for shifting the belt.

An engine in use in a cement works was having quite a hard time. The usual kick about the engine was made by the engineer. Upon investigation it was seen that the crank arm had been broken at some time, and had been patched with side pieces of metal as at D, *Fig.* 4. This resulted in a very unsteady motion of the engine. A new crank arm was sent for and put on.

A choked measurer feed was found in use in a cement works, and the parties engaged at the work claimed that the machine was no good. An examination brought out the fact that a leak had occurred at some time in the feed-pipe, and in fixing a patch over the leak, as at E, *Fig.* 5, the loose metal of the edges of the pipe were turned inward and clogged the passage. The entire patch was removed and the upturned edges hammered smooth. Then the patch was replaced, and the passage was made free.

A coupling of a cement-working machine was found with a series of keys driven as at F, *Fig.* 6. These keys made the hub of the coupling tip upward. The alignment of the shaft was thrown out as a result. In order to properly fix this coupling the keys were driven out, and a new coupling with proper sized bore was put on. Then in another cement-working machine the flanges of a coupling were just barely holding together with the bolts partly loose, as at *Fig.* 6. The bolts were removed, the coupling reset, and then new bolts with nuts properly attached were inserted. One case of imperfect work in a good cement machine due to the working loose of a threaded shaft and union of the design shown in the sectional drawing at H, *Fig.* 7. The threaded shaft was reinserted, and then, to hold the same from turning out as before a pin was put through a hole drilled straight through the exterior hub and the shaft.

RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

BRITISH ASSOCIATION OF MANAGERS OF TEXTILE WORKS.

MODERN MILL CONSTRUCTION.

Paper by MR. F. MOORE.

Below we give an extract from a paper read by Mr. F. Moore (of Messrs. Moore and Crabtree, Architects, Keighley) before the Yorkshire section of the British Association of Managers of Textile Works, at a meeting held at the Bradford Technical College.

After remarking that a large number of the structures used in the Bradford trade were of considerable antiquity, and that any further extension of the wool industry would have to be provided for by the erection of new structures capable of bearing the strain of vibration of the powerful modern machinery, the lecturer discussed the kind of structure which would be necessary.

The development, he said, in the manufacture of constructive steel, and its use in place of cast iron and timber for flooring beams and pillars, had made it structurally possible to cover larger areas free from obstruction by pillars; and the development of concrete reinforced by steel in a variety of forms had of recent years made rapid strides, and it had now practically superseded wood for the construction of all floors above the basement; the cost of the former being to-day not more than 20 per cent. over that of the latter.

When it was first introduced into this country, some twelve or fifteen years ago, its cost was almost double that of wood; but the improvement and cheapening of the cost of constructive steel work, with an increased knowledge of its application in positions where its great tensile strength are brought into full play, together with improvements that at the same time have been made in the manufacture of cement, had all tended to lower the cost of reinforced concrete considerably; while during the same period timber had (alone among the materials used in building) considerably increased in price, so that to-day strength for strength there was only about the percentage of difference above mentioned.

Wooden floors in a worsted mill in the course of years get saturated with oil, greatly increasing the fire risk; and when it was discovered that the behaviour of cement concrete and steel in the matter of expansion and contraction under heat and cold were identical, an extensive field for its use was opened out for fireproofing purposes alone. While the cost was much greater than wood, however, the action of the insurance companies in declining to fully recognise their lessened risk by a corresponding reduction in the premiums, prevented its being used to the extent it would otherwise have been. To-day, however, they did so, and speaking broadly they quote rates which show a reduction of from 20 to 40 per cent. in favour of the best fireproof construction, so that to the insured it now offers advantages on the reduction of fire premiums.

Another great advantage to be obtained was the greater rigidity by which the whole of the upper floors can be constructed so as to withstand if need be the shocks of the squeezing rollers of washing machines, the dabbing motion of a comb, or the swing of a 86-in. coating loom with the same freedom from vibration as a floor on the solid ground.

An additional advantage, too, was that these floors by being carried right through the outer walls will bind together the whole structure in a most effective way and to a degree quite impossible in the days of wooden floors. In wool warehouses it had been found that bales of wool on wagons (and, in fact, anything on wheels) run much more easily.

Last, but not least, might be mentioned immunity from the decay which sometimes occurred in timber quite unaccountably, as instanced by the collapse of a portion of floor at Legrams Mill three years ago, although this floor was strongly constructed of the best material then known.

A disadvantage often urged against the use of concrete floors was the dust arising therefrom in the early years of its existence, and considerable study and a number of experiments have been tried to obviate this drawback. We had found where it exists that two coats of boiled oil laid on the surface was a considerable benefit, but best of all—and this is an almost complete prevention of the dust nuisance—was to have the finished surface made with washed Granite, at a slight additional cost. This made a beautifully hard floor, almost unwearable, and free from dust. The gates and passages might with advantage be boarded on to coke breeze concrete.

It is well known that good cement concrete, while of great strength in compression, possessed little tensile strength, while steel possessed great tensile strength, and that the two materials in combination had the same expansion and contraction under heat and cold. The main points to be observed, therefore, in the use of reinforced concrete floors was that the steel reinforcement should be embedded in its thickness, as near the underside as possible, so as to leave it completely covered and free from risk of corrosion, and the system which most effectually answers this requisite at the lowest cost was the best.

But although this material offers such obvious advantages for use in the flooring of mill buildings, it is not likely to supersede the use of stone and brick in this district for the construction of walls.

The usual—and doubtless the best in this locality—covering for roofs is slate.

The extra heat generated in rooms under blue slate with a hot summer sun had, however, caused considerable inconvenience, and to obviate this it had become common to have such slates whitewashed on the outside, especially in weaving and combing sheds—not perhaps a very expensive operation in itself, but the cause of much damage by broken slates, etc.

We had, however, in recent years successfully overcome this difficulty by using coke breeze concrete slabs under the slated sides of saw-tooth roofs, the constructive ironwork being covered with 2½-in. thick slabs on to which the slate was nailed, thus preserving to a greater degree than ever the old grey slate and equability in temperature of the room underneath.

The roof lights of both the saw-tooth or ordinary roofs were now invariably constructed with one of the patented systems of roof glazing; while the cost was slightly more than the old wood bars and putty glazing, it was not only far more lasting, but gave more light and also saved not only the triennial painting that the latter requires, but also the slate breakages under the old system.

NATIONAL ASSOCIATION OF CEMENT USERS.

UNIT COSTS OF REINFORCED CONCRETE FOR INDUSTRIAL BUILDINGS.

Paper by MR. CHESTER S. ALLEN.

The following paper on "Unit Costs of Reinforced Concrete for Industrial Buildings" was read by Mr. Chester S. Allen, of the firm of Lockwood, Greene and Co., architects, Boston, at the recent Convention of the National Association of Cement Users:—

Unit costs are harmless when used with judgment and prudence, but likely to bring remorse and anguish when employed promiscuously. Rare and talented, indeed, is the man who possesses the experience, judgment and intuitive sense to know when, where and how to properly modify any tables or statements of unit costs to meet the peculiar conditions of each individual case. While the figures given in this paper are all taken from structures erected during the past two years under the writer's supervision, the wide range of territory, local conditions, and different seasons of the year under which the various pieces of work have been executed are so great as to render the information of value only in a very general way.

As a general proposition we have found that reinforced concrete is the lowest priced fireproof material suitable for factory construction, and while it is true that its first cost will generally run from 5 per cent. to 20 per cent. higher than first-class mill construction, we have recently had several instances where, with lumber at a high price, reinforced concrete has worked out cheaper than brick and timber. It is especially adapted to heavy construction, and for heavy loads of 200 pounds per square foot and over, where the spans are 18 to 20 ft. centres, not even timber can compete with it.

TABLE I.

Type of Building	Dimensions	No. Stories	Story Ht.	Live Load lbs. per sq. ft.	Type of Construction	Col. Spacing	Total Cost Per sq. ft.	Total Cost Per cu ft.
							Dols.	Dols
Machine Shop ...	120′ × 50′	4	12′	150	Beam	10′ × 24′	1·17	0·09
Cotton Mill ...	550′ × 129′	2	16′	75	Beam	10′ 8″ × 25′	0·98	0·07
Factory	400′ × 52′	4	12′ 6″	150	Flat Slab	17′ × 20′	1·09	0·077
Weaving Mill ...	140′ × 60′	5	12′ 6″	150	Flat Slab	17′6″ × 20′	1·50	0·12
Knitting Mill ...	220′ × 75′	2	14′	125	Beam & Girder	22′ × 25′	1·09	0·073
Cartridge Factory	223′ × 56′	2	16′	300 & 1000	Beam & Girder	18′ 6″ × 18′ 6″	1·55	0·10
Weave Shed ...	341′ × 231′	1		125	Sawtooth Skylights	13′ × 21′4″	1·79	0·07
Machine Shop ...	220′ × 100′	1			Sawtooth Skylights	20′ × 20′	1·75	0·10
Store House ...	181′ × 56′	4	14′ 6″	150	Flat Slab	18′ × 20′	1·15	0·07
,, ...	580′ × 109′	10	12′	250	Beam & Girder	19′ 3″ × 19′ 25′	0·85	0·071
,, ...	580′ × 244′	4	15′	350	Flat Slab	18′ 6″ 25′ } × 25′	0·76	0·05
,, ...	256′ × 100′	12	8′	150	Flat Slab	16′ × 16′ 8″	1·04	0·12

The unit costs of projected or completed buildings are commonly figured either as so much per cubic foot or as so much per square foot of area occupied. Table I. gives the unit costs both on the square foot and the cubic foot basis, together with a general description of a number of reinforced concrete industrial buildings of different types erected during the past two years. It will be seen from an examination of this table that the average cost per square foot of these buildings, excluding the one-story structures, was $1.12; while the average cost per cubic foot was 8·7 cents. The one-story structures both had reinforced concrete sawtooth roofs, and the average cost per square foot was $1.77, while 8·5 cents was the average cost per cubic foot. The above costs are for the finished buildings, including plumbing, but do not embody heating, lighting, elevators, sprinklers and power equipment. The cost per square foot of floor area was obtained by dividing the cost of the building by the total number of square feet of floor area, exclusive of roof area, but including basement floors; and the cost per cubic foot, by dividing the cubical contents into the cost of the structure.

While no coal pockets are included in Table I., it has been our experience that above 3,000 tons capacity reinforced concrete elevator coal pockets cost from $5.50 to $7.50 per ton of capacity. Standpipes, exclusive of the foundations, average from 2½ to 3 cents per gallon of capacity.

On much of the reinforced concrete work which has been done under our supervision it has been possible, owing to the contract being either on a percentage or cost plus a fixed sum basis, for us to obtain quite accurate and comprehensive cost data. This data, of course, are only of particular value when all the local colour of each specific case is known, but the average results are at least interesting.

The average unit cost of the 1 : 2 : 4 concrete in the floors, including the beams, girders and slabs, was $6.10 per cubic yard, and for the columns $6.70 per cubic yard. Where 1 : 1½ : 3 mixture was used for the columns, the average cost was $7.60 per cubic yard. This cost was made up of the items of cement, sand, stone or gravel, labour and plant. The cement, of course, varied greatly with the demand, but the average net cost was $1.35 per barrel, including 3 cents for tests. The sand averaged 80 cents per cubic yard, and the crushed stone $1.25 per cubic yard. The cost of labour of unloading the materials and mixing and placing the concrete varied from 65 cents to $2.90 per cubic yard. The cost of plant, consisting of freight, depreciation or rental of mixing and hoisting towers, erection of same, power and coal, and losses and waste on the small tools, ranged from 50 cents to $1.50 per cubic yard of concrete placed.

Next to the proper design of the structural features of a concrete building, the economical design of the form work is of paramount importance. The truth of this statement is borne out by the fact that on the average job the cost of the forms amounts to about one-third the cost of the entire structure. On the buildings under consideration the average cost of the forms for the floors, including beams, girders and slabs, was 10 cents per square foot, and for the columns 13 cents per square foot. The lowest cost was in a building of the girderless or flat slab type of construction, where, by the intelligent use of corrugated iron for the slab forms, the cost of the floor forms, including wall beams, was 7 cents per square foot. The highest cost was for an artistic, but not elaborate, overhanging cornice on a twelve-story building, and was 32 cents per square foot. This last item rather forcibly demonstrates that any attempt at architectural development is very apt to be a costly proposition.

The cost of the labour of making, erecting and stripping the forms varied according to the price of lumber, design of the structure, method of forming, character of the supervision, and the skill of the workmen from 4¾ to 12 cents per square foot. The cost of lumber, nails and oil, divided by the square feet of forms, averaged from 2¼ to 4½ cents per square foot.

The cost of bending and placing the reinforcing steel, including the necessary wire, averages $10 per ton, the range being from $5.75 per ton to $17.20 per ton.

Granolithic floor finish 1¼ in. thick, when laid before the concrete below it had set so as to form one homogeneous slab, cost on the average of 4½ cents per square foot. When put on after the rough concrete slab, the cost averaged 7 cents per square foot.

Inasmuch as the only economical design of a reinforced concrete structure is one which closely resembles that of the steel skeleton type, the relative cost of the various materials commonly used for curtain walls under the windows may be of interest. The writer has used brick, vitrified tile, concrete blocks, cast concrete slabs, and solid concrete walls for this purpose.

The most common type of curtain wall has been either an 8-in. or 12-in. brick wall resting on the concrete wall beam. The average cost of these walls has been 45 cents per square foot. There is practically no difference in cost between the 8-in. and the 12-in. brick curtain wall, as the saving in material is offset by the great amount of extra labour in culling and laying the thinner wall.

An excellent and inexpensive curtain wall is constructed by using 8 in. × 12 in. × 18 in. vitrified tile. This is a non-absorbent wall, and when properly laid in cement mortar makes a tight weather-proof curtain wall. The cost of this wall averages about 25 cents per square foot. If the tile is plastered both sides the cost is about 38 cents per square foot.

Where 8-in. concrete curtain walls were cast in place after the skeleton frame was completed, the average cost was 40 cents per square foot, and when poured simultaneously with the columns 48 cents per square foot. Four-inch cast concrete slabs cost about 35 cents per square foot.

While concrete blocks make a very cheap and light curtain wall, the price being about the same as for the 8-in. tile, the writer's experience with them has been rather unfortunate on account of their extreme porosity.

Where the location of the buildings have demanded special treatment of the exposed surfaces we have generally specified rubbing with a block of carborundum. The average cost of this work has been 4 cents per square foot. In two instances portions of the structures have been bush-hammered, with a resulting average cost of 7 cents per square foot.

Concrete piles were used on the foundations of several of the buildings, and the average cost of the piles was $1.15 per linear foot.

The most common methods of waterproofing concrete structures are by the introduction of foreign ingredients into the concrete, by the application of a compound to the concrete surface, by the use of paper or felt waterproofing, and by accurately grading and proportioning the aggregates and the cement.

Where an addition of hydrated lime in the proportions of 10 per cent. to the weight of the cement has been used, the added cost to a cubic yard of 1 : 2 : 4 concrete has been 50 cents. Patented compounds have cost from 25 to 35 cents per square foot of surface covered. On horizontal or inclined surfaces we have sometimes used a granolithic surface of rich mortar of Portland cement and sand, or Portland cement and screenings, in the proportions of 1 : 1 laid at the same time as the base, and trowelled as in sidewalk construction. The cost of this work has been about 5 cents per square foot.

Taken as a whole, the lowest possible cost on a reinforced concrete building can be obtained only by a careful study of each particular case to determine the cheapest type of construction and most economical spacing of columns. As a general proposition we have found that for light loads with ordinary beam and girder construction the most economical spacing of columns has been 18 ft. each way, and for flat slab construction 20 ft. each way. For heavy loads, such as 300 pounds per square foot and over, it has been our experience that the cheapest column spacing for beam and girder construction is 15 ft. × 15 ft., and for flat slab construction 17 ft. × 17 ft. In arriving at the most economical layout, it is always well to bear in mind that the construction which allows the greatest simplicity of form units, together with the maximum number of repetitions of same, is invariably the one that will work out cheapest in the end. The fact that the actual amount of concrete or steel required for a certain floor construction is less than that required in another by no means implies that this is actually the cheapest floor construction, as the unit labour of the form work may easily have been increased out of all proportion.

MICHIGAN COLLEGE OF MINES CLUB.

REINFORCED CONCRETE AS APPLIED TO MINE SHAFTS.

Paper by MR. E. R. JONES.

The following paper was recently read before the Michigan College of Mines Club, Houghton, Michigan, by Mr. E. R. Jones:

This evening, as at another time, I find that I am acting in the capacity of lieutenant of Mr. Uren. Mr. Uren has already gone into the subject of concrete construction generally, and in some detail, and it has been left to me to furnish some information on reinforced concrete as applied to shaft construction.

I wish to thank Mr. S. R. Smith, of the Ahmeek Mining Company, and Mr. Will Smith, of the Mohawk Mining Company, for information furnished me.

That which follows deals principally with the making and installation of concrete, and most of the information which I may be able to furnish you has been acquired through personal experience and intimate contact with the work.

Since the cost of material for the making of concrete varies widely with the locality and the property, and there is also a discrepancy in the wage scales of the different mining companies, any detail of costs would not only prove tiresome but would be of little value, except where conditions were exactly similar to those below described, so that, where cost is mentioned, it will be for the purpose of comparison in a special case with timber which the concrete has supplanted.

For a number of years solid concrete and reinforced concrete shaft collars and shafts have been in vogue where the conditions warranted a shaft of any degree of permanence, but not until 1909 was reinforced concrete tried as a substitute to take the form and similar methods of installation as the long-used timber sets for shaft purposes—namely, at the No. 3 and 4 shafts of the Ahmeek Mining Company.

In the beginning, two distinct kinds of material were used; a good grade of gravel and natural sand from a local pit, and the trap rock, through which the shafts were sinking, together with clean conglomerate sand from the C. and H. Mill. Sets were moulded from these two classes of material and installed with equal partiality, and subsequent service has proven both to be equal to the demands made upon them. Pieces set aside for the purpose were allowed to season sufficiently that they might be given a fair competitive test, and it was found, on comparing the fractures in the two combinations of material, that the sand and cement filling the spaces between the rounded pebbles broke away from them, while the fracture in the trap-conglomerate same combination continued through the larger elements of the mixture. The gravel mixture could doubtless have been improved considerably by careful washing, but the cost of preparation, compared with the trap rock and conglomerate sand, prohibited its use in this particular case.

The materials finally used were as follows:—

No. 1 Portland cement, conglomerate sand, trap rock trommelled over $\frac{3}{4}$ in. through screens.

The proportions used were 1 : 3 : 5 in wall plates, end plates, and dividings, and 1 : 2 : 4 in studdles. The reinforcement in wall and end plates consisted of three $\frac{3}{4}$-in. Monolith steel bars with $\frac{1}{4}$-in. webs, crimped on to them, together with two straight $\frac{3}{4}$-in. Monolith bars. The dividings were reinforced by four $\frac{1}{2}$-in. Monolith steel bars wound spirally with $\frac{1}{4}$-in. steel wire, the whole presenting a column with square cross-section. Studdles were reinforced with two pieces of old wire rope $1\frac{1}{4}$ in. in diameter. Reinforced concrete slabs were moulded for the shaft lining, the material used being fines of trap rock under $\frac{3}{4}$ in., conglomerate sand and Kahn expanded metal as reinforcement. The mixture used for slabs was 1 : 2 : 4. By way of experiment, the writer selected a piece of No. 1 hemlock plank of the same length, width and thickness of a concrete slab, which had seasoned for one year, supported them at either end, and placed them side by side, and then applied an equal pressure across the centre of each. Three failure cracks appeared in the concrete slab just previous to the breaking of the hemlock plank, although total collapse of the concrete slab did not occur until the pressure was considerably increased. While the method of the test employed was crude, it proved to the satisfaction of the writer that the concrete slab was much superior in strength. Considering the rapid decay of timber used as shaft lining, no further comparison of the two is necessary.

In the moulding of the concrete sets, 2-in. No. 1 white pine was used in the construction of the forms. These were soaked in Delaney's wood preservative, and repainted with preservative on the interior each time before setting up, thus insuring them against warping and prolonging their lives indefinitely, as well as securing a smooth and easy parting from the concrete when removed. A Smith barrel type mixer was employed in preparing the charge for the forms. The amount of water used in the mix was such that, when the batch was piled, it settled rapidly without agitation. A drier mix was attempted by way of experiment, but, due to the amount of reinforcement employed, it was found impossible to ram the drier mix into place.

The labour involved in the making consisted of two carpenters setting up forms and keeping them in repair; one man wheeling forms on to skidways ready for filling, returning used forms to shop and cleaning the same; one man feeding mixer from stock piles of rock, sand and cement; one man delivering mix to forms and shovelling material into place; and one mason ramming charge into final position. With this combination of men, as many as four complete sets, consisting of 64 separate pieces, have been moulded in one day of nine hours. In ordinary weather the sides of the forms were allowed to remain in position over-night, and then removed, while the bottoms were left in place another twenty-four hours. The bottoms were removed by turning the pieces on their sides, where they were left to harden one day longer before removal to the stock pile. All through the process of removal the sets were handled with the greatest care in order to preserve the appearance of the set and prevent cracking, which might not develop to the eye until weathered. All skidways used in making and storing were brought to a level to prevent warping and bending while the sets were green, to insure a perfect fit underground, for, unlike timber, the concrete set cannot be brought to place unless perfectly true. Sets should not have been used under sixty days after removing forms, although we, through the reduction of the stock piles, have been forced to install pieces of fourteen days set, but the greatest care was observed in handling and putting in place underground. Concrete sets one year old, which have been subjected to all manner of weather, can be abused somewhat and handled almost as carelessly as timber.

As before stated, the above-mentioned sets were made for the No. 3 and 4 shafts of the Ahmeek Mining Company. The shafts are of the three compartment variety—two skipways and one manway, dipping at an angle of 80 degrees. The outside dimensions of the compartments are:—

Skipways—7 ft. 6 in. high; 6 ft. 10 in. wide.

Manway—7 ft. 6 in. high, 3 ft. 0 in. wide, with the end plates and dividings, making the greatest span of 7 ft. 6 in. Offsets were moulded in all plates 5 in. from the inside face to accommodate lining slabs. Also holes were cored for the use of hanging bolts and bracket bolts. The wall plates, end plates and studdles have a cross-section of 80 square in.; dividings 81 square in. The percentages of reinforcement are approximately as follows:—

Wall and end plates	5 per cent.
Dividings	5 ,,
Studdles	3 ,,

It was found advisable from the beginning, because of the great weight of the wall plates, to mould them in two sections, one section spanning the ladder way, and one skipway, and the other section spanning the remaining skip compartment. These two sections were connected when in place by two bolts passing through holes, cored for the purpose, and two straps of iron spanning the splice. Studdles were made for 4 ft. 0 in., 5 ft. 0 in., and 6 ft. 0 in. sets to accommodate the ground passed through.

The weights of the different pieces comprising the set are as follows:—

Long section of wall plate	1,035 lb.
Short section of wall plate	700 lb.
End plate	600 lb.
Divider	645 lb.
3 ft. 3 in. studdles	268 lb.
Complete set of 16 pieces	8,104 lb.

Taking the weight of No. 1 western fir, which has been exposed to the weather in stock piles, as 33 lb. per cubic feet, the above concrete set weighs almost three times that of a 12 × 12-in. timber set, which the concrete set is intended to replace. Because of this additional weight of the concrete set, it was found necessary to increase the usual five or six men on the timber gang to seven in number. In a vertical shaft, to which the concrete sets are especially adapted, the number of men per gang might again be reduced. The sets are hung or built as the ordinary timber sets, only requiring an additional rope and block to swing the pieces in place. After the sets are wedged to line, bottoms are put in between the plates and the surrounding shaft wall, and the set is then tied to the shaft wall by means of concrete in the proportion of 1 : 3 : 5. The concrete slabs are then put in place, and loose rock thrown behind them, filling up what space still remains between the set and the wall of the shaft.

After the set is in place, it is extremely important that it be well protected from the blast, for, unlike the timber set, concrete will not stand the blast. For this purpose the writer used flat timber and steel plates chained to the underside of the plates and dividings, and even this precaution was at times inadequate. Where the ground was breaking easily the sets have been as near as 12 ft. to the miners, and again, when the ground was especially refractory, sets 40 ft. from the blast have been cut out. It is obvious that it is well to keep as far behind the mining as the ground will permit. In dangerous ground, which required timbering close up to the sinking, timber sets were used, but, had not time played an important part in the sinking, no ground was met in which concrete sets could not have been installed. With a gang of seven men one complete set can be installed in a nine-hour shift. This permits a sinking rate of better than 100 ft. per month, which was accomplished at the No. 3 and 4 shafts.

The comparative cost of the concrete set and timber set, delivered at the shaft collar, is striking. The concrete set was delivered for $22.50, the timber set for $37.60.

The Ahmeek Mining Company, I believe, was the first to adopt the concrete stringers, and the Mohawk Mining Company soon followed with their use. At the Ahmeek these stringers have been in continuous use since the beginning of operations, and have required no repairs. Supt. Smith, of the Mohawk, has informed me that soon after the stringers were installed skip repairs increased about 100 per cent. The stringer being entirely rigid and the skip also of rigid construction, the axles of the skips were found to be crystallised and the rivets working loose. This feature was overcome by moulding 2-in. pine strips, after preserving them in Delaney's Wood Preservative to prevent decay, into the stringers at intervals of 3 ft., allowing them to project ½ in. above the face of the stringer, and resting the rail thereon. The pine strips have been in place four years, and none has been replaced to date, and skip repairs have been reduced to normal. Possibly because of a differently constructed skip, Ahmeek repairs were not abnormally high, but the same racking of the skip body occurred, and the Ahmeek Company has adopted the Mohawk feature, and expects to profit accordingly.

Concrete plats, or stations, have been in use at both the Ahmeek and the Mohawk for some time. They differ from the timber plat in outward design only in the cross-section of the members, which are 9 × 12 in., and are reinforced with old rail and wire rope, and replace the 12 × 12-in. and 12 × 14-in. timber formerly used. Holes are cored to accommodate gates for skip and dump doors, and tram rails are embedded in the concrete, making the use of spikes unnecessary. When turn-tables are used on the back of the plat, the rigidness furnished by the concrete insures the trammers against derailed cars, resulting from a tilted table.

At the present time our company is installing reinforced concrete dividings to replace the practice of putting in 10-in. flat timber. In cross-section they are 9 × 12 in., and are reinforced by old rail. On the ladder road they are placed 6 ft. from centre to centre, and between the skip compartments are put in as often as the hanging requires. Since the casing along the ladder road performs no other office than the protection of the men while on the ladder, or in case of a fall, plank is used for the purpose, and a 3-in. hemlock strip is moulded into the dividings to facilitate the fastening of this casing.

Quite often, in the placing of concrete and reinforced concrete, both above and below ground, not enough attention is paid to the character of the men employed in charge of the mix and actual distribution of the material. It is not enough that the work shall look finished and neat on the removal of the mould boards, which any gang of men can accomplish with only this end in view. The placing of concrete where strength is desired, as well as weight and finish, requires the greatest care and judgment. Men should be selected who will see that the fines are uniformly distributed with the coarser material, for, unless the rock of the mixture is made to well overlap, congregation of coarse material and fines will accumulate which will result in a weakness which often cannot be detected after the work is completed. The ideal method of placing the mix is by hand with shovel, but in shaft work this method is slow, and requires extra labour where the work is situated some distance from the place of mixing. Where chutes are used to convey the mix to its destination, the larger material arrives in advance of the fines, making an even distribution difficult and at times impossible. The writer has eliminated this feature by placing traps at regular intervals in the conducting launders for the purpose of retarding the larger particles, thus securing a more even mix at the end of the launders than at the beginning.

Concrete has long been used underground for bulkheads, forks, open gutters and casings for fire doors, and cannot be surpassed for these purposes. As applied to shafts, the material is comparatively new, but each succeeding year marks its advance, and in the end timber will have been entirely superseded. For much of the underground construction timber is still the rival of concrete, but, due to the increasing scarcity of the timber suitable for mine use, it cannot long remain as such, and must soon make way for the more plentiful materials, concrete and steel.

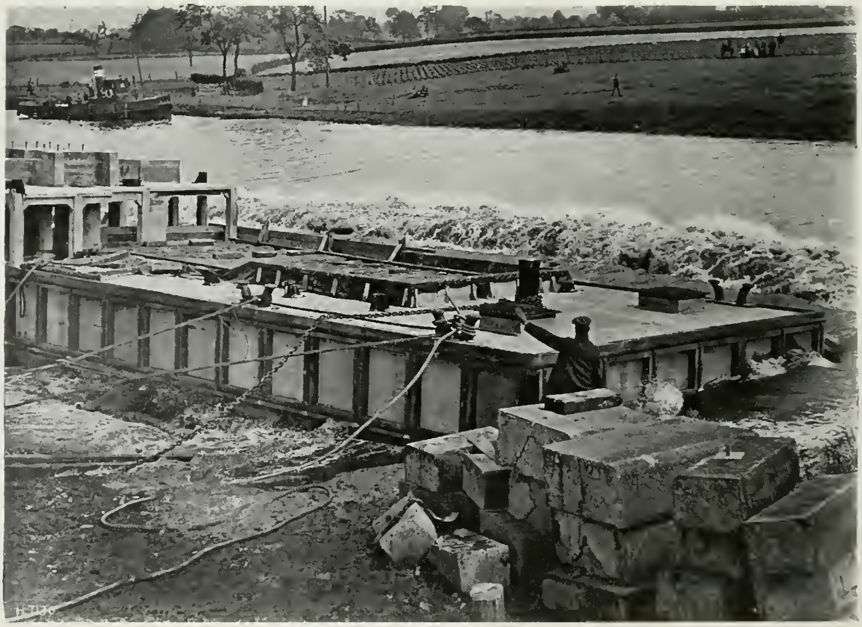
NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

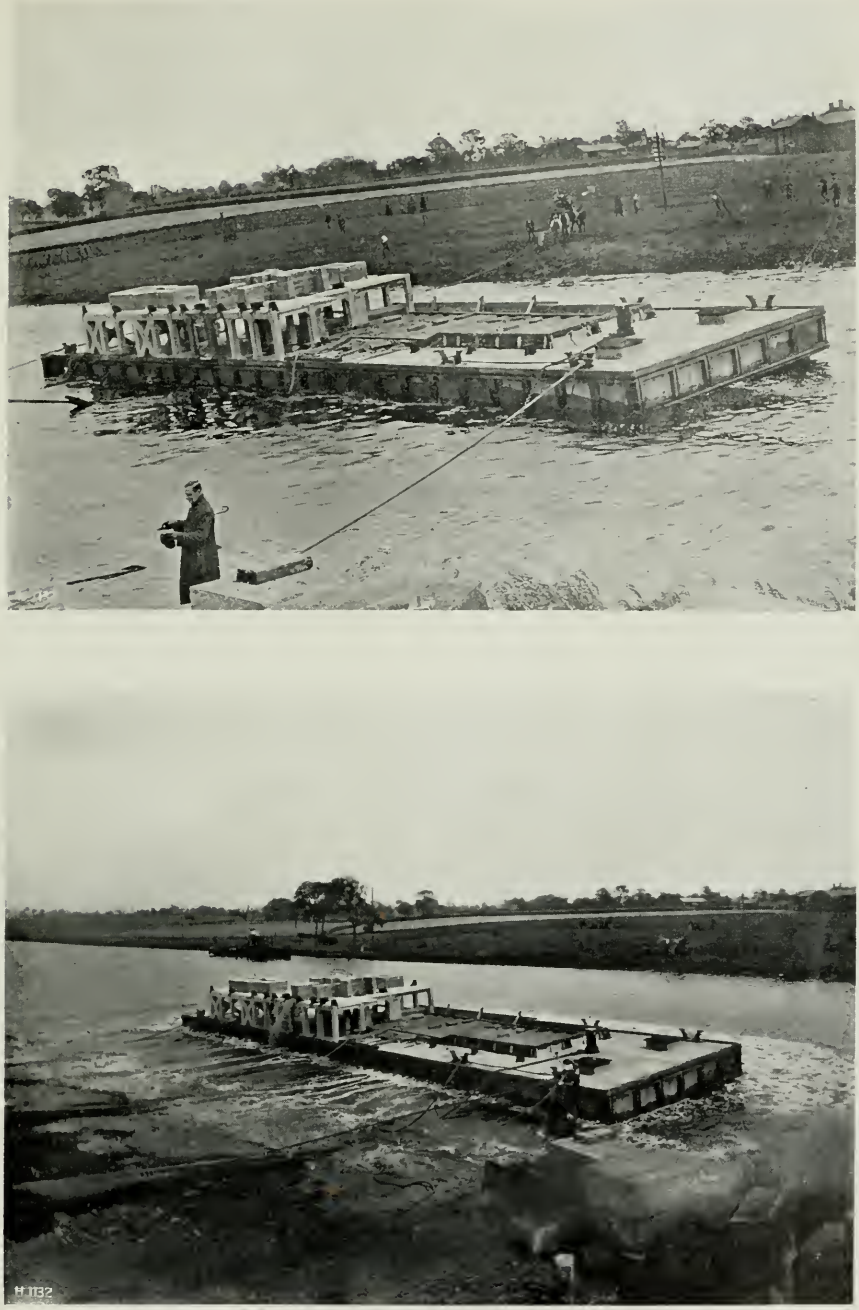
REINFORCED CONCRETE PONTOON AT IRLAM, MANCHESTER SHIP CANAL.

THE reinforced concrete pontoon constructed and launched at Irlam for use on the Manchester Ship Canal marks a further advance in the application of reinforced concrete to shipbuilding; and, apart from this, the vessel embodies principles that deserve the consideration of those who, in view of the *Titanic* disaster, are seeking to improve the design of unsinkable ships.



Reinforced Concrete Pontoon, Irlam.
MANCHESTER SHIP CANAL.

The pontoon was built from the designs of Messrs. L. G. Mouchel & Partners, Ltd., of Westminster, by the Yorkshire Hennebique Contracting Co., Ltd., of Leeds, in accordance with the requirements of the engineers to the Manchester Ship Canal



Reinforced Concrete Pontoon, Irlam.
Manchester Ship Canal.

Company. Its construction was authorised by the late Mr. Hubert Congreve, M.Inst.C.E., and continued under the direction of Mr. H. A. Reed, M.Inst.C.E., the present chief engineer to the company.

The vessel measures 100 ft. long by 28 ft. beam by 8 ft. 6 in. deep from the main deck to the keel, and will draw about 6 ft. 6 in. of water when fully loaded.

The mechanical equipment includes a steam boiler of the Scotch marine type, 13 ft. 8 in. in diameter by 10 ft. 6 in. long, weighing 58 tons; a compound vertical steam engine, condenser, three powerful centrifugal pumps, and three steam winches. The machinery represents the weight of some 43 tons, and, being placed at one side of an upper deck, will be counterbalanced by water ballast in water-tight compartments on the opposite side of the vessel.

It is interesting to know that the adoption of reinforced concrete in place of steel for the construction of the pontoon was influenced mainly by the lower cost of reinforced concrete, the avoidance of maintenance charges, and the greater ease with which repairs could be conducted in case any damage should be caused by collision or other accident.

Although the risk of injury to the vessel is somewhat remote, the construction of the hull is such that, even if the exterior shell were completely destroyed, the main part of the pontoon would still remain afloat.

The inner portion comprises one large water-tight compartment for the coal bunkers, stokehold and boiler plant, and four smaller compartments for ballast and for use by the staff. Around these five compartments are sixteen perfectly independent water-tight compartments, forming a series of cells approachable only by way of hatches and manholes from the main deck. Thus in the event of a collision the efficiency of the bulkheads would not depend upon the closing of water-tight doors, as in ordinary practice.

The framework of the pontoon consists of beams, ribs, columns, and struts, all in monolithic connection and further connected by the longitudinal and transverse partition walls, or bulkheads, the bottom, sides and ends of the hull, and the main decking slab. Some of the columns are continued above the main deck for the support of an upper deck on which are foundation beds for the steam engine and pumping machinery.

The compartment devoted to the boiler plant contains a very powerful beam supporting the sloping floor of the coal bunkers and three transverse cradles carrying the steam boiler. Apart from these members the construction is remarkably light, and the outer skin of the hull is only 3 in. thick.

In autumn last year the moulds were laid for the bottom of the vessel. After supplies of materials had been collected, concreting was commenced early in November, and by the middle of the succeeding month the work had been carried up to main deck level. In January this year the concreting of the main deck was started and practically finished by the end of the month. Then followed the moulding of the upper deck columns, and by March 9th all the reinforced concrete work had been finally completed. The engine and pump beds in plain concrete were finished a week later. All the water-tight compartments were carefully tested by filling them with water, an operation which was completed in May last.

EXPANDED STEEL CONCRETE CONSTRUCTION AT 30a WIMPOLE STREET AND 32 NEW CAVENDISH STREET, W.

The rebuilding of the above premises for Dr. Irwin Moore, carried out to the designs and under the supervision of Messrs. Banister, Fletcher & Sons, architects, 29, New Bridge Street, London, E.C., afford a good example of the application of expanded steel-concrete for interior construction.

The building, which consists of two houses divided by a party wall, occupies a prominent position at the north-east corner of the junction of the two streets, there being a frontage of about 23 ft. in Wimpole Street and about 90 ft. in New Cavendish Street.

The following is a brief summary of the work.

There are five suspended floors (ground-floor to fourth floor) to each house.

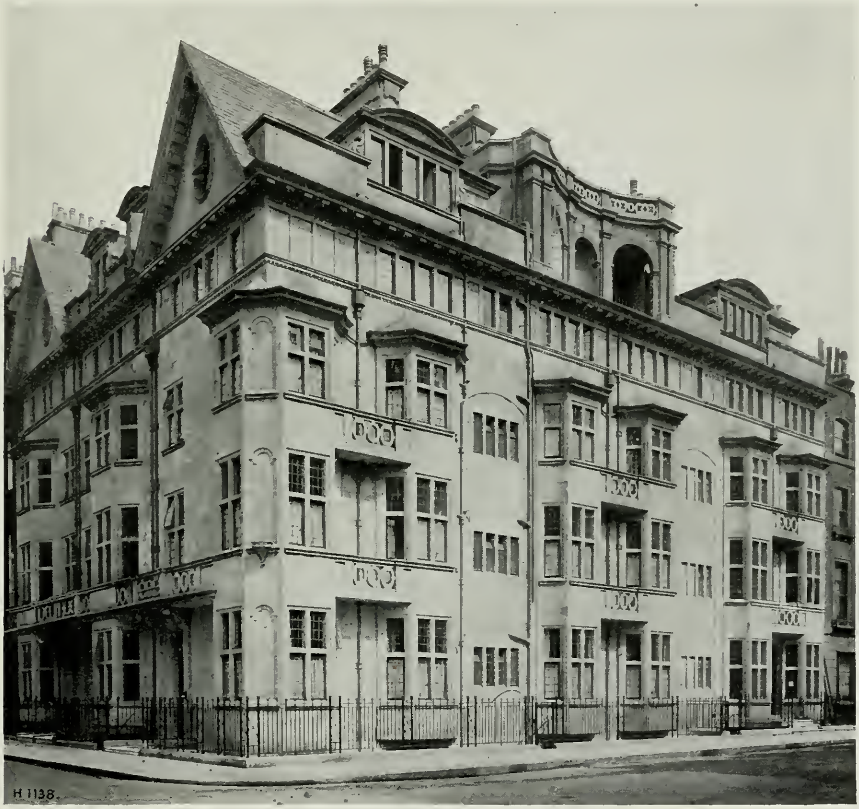
The roof, which is partly flat and partly sloping, runs into a gable wall at the

Wimpole Street front, and contains several dormer windows on the Cavendish Street front.

The floors and roof are constructed of concrete 5 in. thick, except the sloping portions of the roof, which are 4½ in. thick, reinforced with expanded steel throughout the underside and at the top side over intermediate supports.

The beams under the floors are generally spaced to come over the light partitions separating the rooms and corridors; these beams are 9 in. deep and 4½ in. wide. An exception is the beam at each floor in the centre of the front room facing Wimpole Street, this being 5 in. deep under the slab and 10 in. wide.

The staircase in the Wimpole Street house is contained partly by two chimney stacks and partly by concrete walls 4½ in. thick reinforced with expanded steel. The steps and landings are monolithic; the steps have a minimum thickness of 4 in. under the oak treads, and the landings are 5 in. thick. Rib mesh expanded steel is used as reinforcement, the longitudinal ribs spanning from side wall to side wall and from back wall to front beam. These stairs are constructed with a rectangular lift-well in the centre.



View of Premises at 30a Wimpole Street and 32 New Cavendish Street.

The staircase in the New Cavendish Street house is of similar construction, with the exception that it is contained on three sides by expanded steel-concrete walls 4½ in. thick and on the remaining side by the party wall of the adjoining building in Wimpole Street. The lift enclosure in this instance adjoins the staircase against the party wall, and its walls also are of concrete 4½ in. thick reinforced with expanded steel.

The floor plans are generally similar to one another, the height from floor to floor varying from about 11 ft. to 9 ft. in the clear. Lavatories adjoin the staircases in both houses between the floor levels.

At the fourth-floor level in the Wimpole Street house there is a belvedere facing New Cavendish Street; this, with a 9-in. brick wall running at right angles to it, is carried by reinforced concrete beams, which were formed above instead of below the floor slab in order to leave a flush ceiling to the rooms below.

The roof is specially designed so that a maximum wind pressure on its sloping surface will not cause any thrust on the party wall of the adjoining Wimpole Street building.

The beams in the floors and roofs are reinforced with mild steel bars, and where such are insufficient to take the shearing stresses the beams are further reinforced. Round mild steel bars are also used in those parts of staircase walls which were constructed to act as beams.

View of Retaining Wall on East Side of Methil Dock.

One of the objects kept in view in designing the interior of the building was to make the utmost use of all the available space.

As most of the beams are constructed over partitions there is practically no obstruction between walls to light and air. The staircase would have been very much heavier in stonework, in which material it would have been difficult to obtain sufficient headroom for some of the lift entrances.

The only structural steelwork used in the building consists of a few solid steel columns in the bay windows along the New Cavendish Street front, and these are encased in expanded metal lathing and plaster; and there are also two beams supporting the brickwork over openings.

All the lintels are made of reinforced concrete.

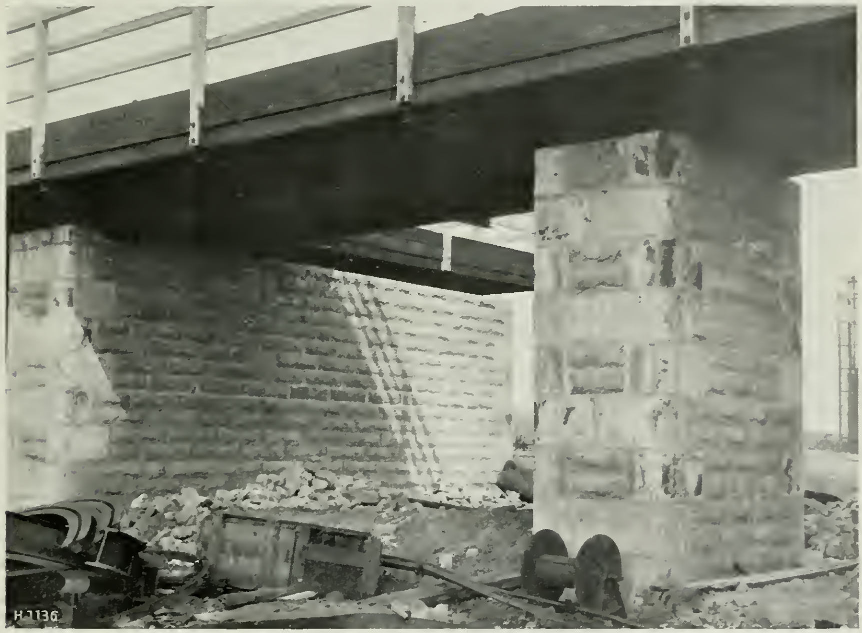
The ground floor of the building is about 2 ft. above street level, and along the frontage is an area to the depth of the basement floor. Under the pavement are a series of vaults, which in general have been left intact during reconstruction, excepting where alterations were required.

The whole of the work was carried out by the general contractor, Mr. A. Monk, Lower Edmonton, London; the reinforced concrete work having been done by the Expanded Metal Company, Ltd., of London, from whom the whole of the reinforcement was purchased.

WORK AT METHIL DOCK.

THE accompanying illustration shows a retaining wall, 400 ft. long, 12 ft. high above surface of ground, on the east side of Methil Dock. It supports a heavy bank carrying railways to coal hoist.

The wall is of concrete faced with "Winget" blocks, the concrete backing and block work facing being brought up together, thus doing away with the use of shuttering to face.



VIEW OF PIERS AT METHIL DOCK.

The second illustration gives a view of piers faced with "Winget" blocks, supporting girders carrying railways to coal hoists.

The contractors for this work are Messrs. Robt. McAlpine & Sons, of London and Glasgow.

CONCRETE IN THE GARDEN.

WE have frequently given examples of concrete used for ornamental and garden purposes, and herewith we give some further illustrations. The three illustrations refer to work carried out at La Mancha Hall, near Southport. *Fig.* 1 is a replica of a seat in the Vatican grounds. *Fig.* 2 is a garden wall built with blocks faced on both sides 14 in. wide with an ornamental gateway. *Fig.* 3 represents the wall on the outside. The whole of this work was carried out by Mr. John S. Rigby, of Liverpool.

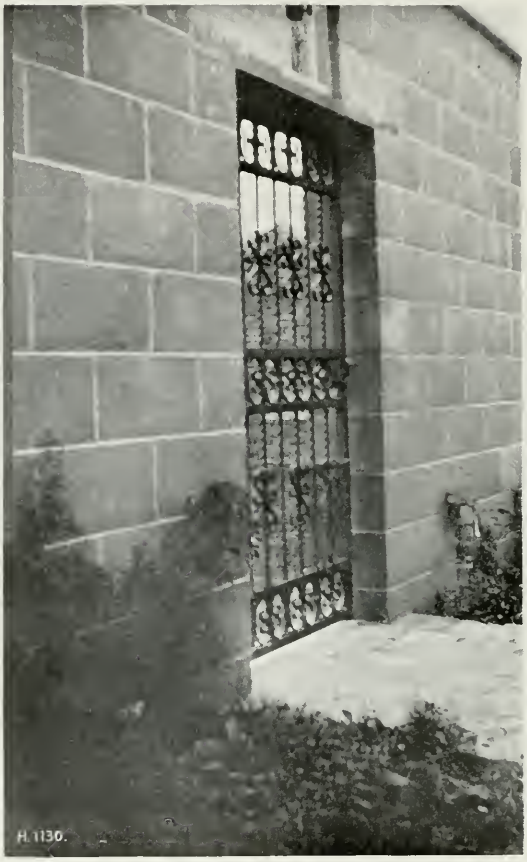


Fig. 3. View of Garden Wall at La Mancha Hall, Southport.

CONCRETE PARAPET RAILING AT TYNEMOUTH.

The accompanying illustration shows a parapet railing which has been erected at Tynemouth under the direction of Mr. John F. Smillie, Borough Surveyor. The railing is 500 ft. in length. The height of the standard, exclusive of tail piece, is 3 ft. 4 in., and the length of the side of the square plinth 7 in. The shaft tapers from a diameter of 3½ in. at a height of 9 in. from the ground to one of 3 in. at the top. The standards are reinforced with ¼ in. round steel rods and spiral wire. The rails are barely 2 in. in diameter, and are reinforced with four $\frac{3}{16}$-in.

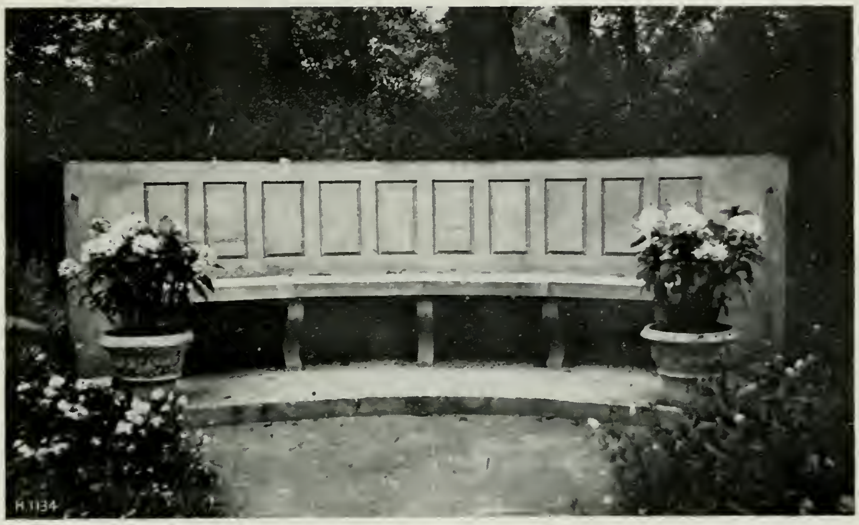
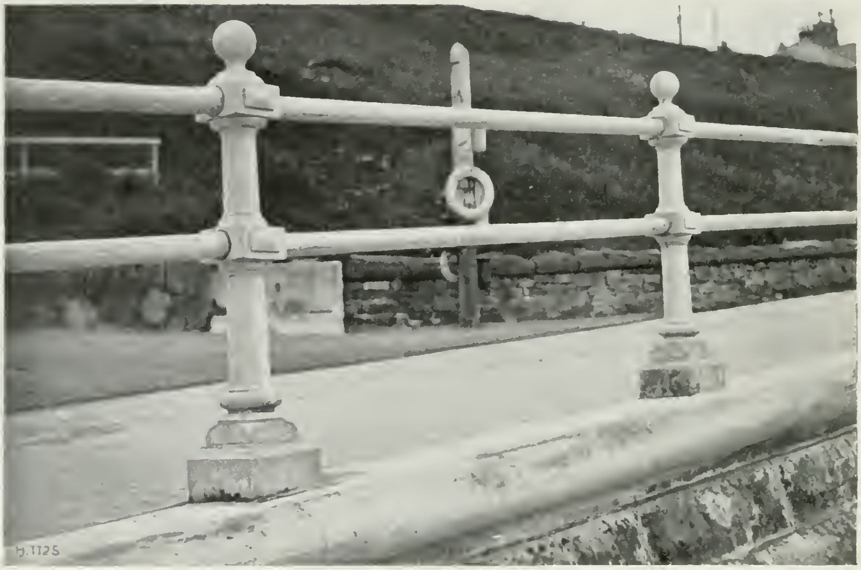


FIG. 1. REPLICA OF SEAT IN THE VATICAN GROUNDS.

FIG. 2. VIEW OF GARDEN WALL AT LA MANCHA HALL, SOUTHPORT.

steel rods and spiral wire. The rail has been up for two winters now, and, although very much exposed to the spray from high seas, it is as good and substantial as when first erected.



Concrete Parapet Railing at Tynemouth.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

" Building Structures in Earthquake Countries." By Ing. Alfredo Montel.

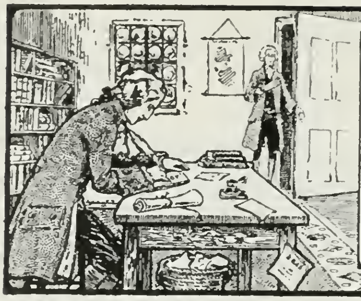
London: Charles Griffin & Co. Ltd., Exeter Street, Strand. Price 8/6 net. 128 pp. + x.

Contents.—On Earthquakes in General—Seismic Action and Nature of the Soil—On Seismic Buildings in General—Abstract of Japanese Rules for the Construction of Earthquake-proof Wooden Houses—Japanese Experiments and Investigations Regarding the Resistance of Brick Columns—Calculation of Brick Columns and Walls of Uniform Strength by Bending—Calculation of Walls of Reinforced Concrete—Freewall Houses—On Monolithic Houses in General—General Test Calculation of a Monolithic Brick Building—Some Notes on the Construction of Masonry—On the Stability of an Ordinary House during an Earthquake—General Test Calculation of a Monolithic House of Reinforced Concrete—Standard Rules for the Execution of Works in Reinforced Concrete.

This work has been translated from the Italian by the author, various additions having been made at the same time, owing to certain particulars being available regarding the earth- and sea-quake of Calabria and Sicily, 1908, which were unknown at the time of publishing in Italy. One of the chief features which will be of interest to engineers of to-day is the portion devoted to reinforced concrete construction, and although our own buildings do not require to be designed to resist seismic forces, we may yet be called upon to design structures for those countries wherein such disturbances take place, and, should previous experience be lacking, we shall welcome a volume which deals with this class of building. In designing a structure which may be called upon to resist earthquake shocks there are two general methods that may be followed—viz., either the different members of the building may be calculated so that each one resists on its own account or the whole building may be calculated as a homogeneous entity acting as a monolith. Generally speaking, the second method is the more reasonable one, and reinforced concrete may be said to be practically the only material that renders this possible and at the same time gives perfect security against fire.

The author expresses the opinion that reinforced concrete is the most suitable material, but he points out the difficulty of finding workmen competent to do this kind of work in places which are often, from the economic and industrial standpoint, very little advanced. There is no doubt that this difficulty does exist, and it becomes highly important that work in such parts should be efficiently supervised by a reliable and intelligent man.

The calculations for the buildings are interesting, and the deductions should be of value, particularly with regard to reinforced concrete work. The volume is well written and very interesting, even to those designers who may never actually require to make use of the recommendations, and it should be worthy of becoming an addition to any engineer's or architect's library of technical works.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

New Bridge for the Lower Ganges.—In a recent number of the *Engineer* it is reported that one of the most interesting, and by far the most costly, of all engineering schemes now in hand in India and the Far East is the new railway bridge to carry the main line of the Eastern Bengal State Railway over the river Ganges at Sara Ghat, which is about 118½ miles distant from the Sealdah terminus, Calcutta. Hitherto the journey from Calcutta to the Himalayas has been made by the vestibuled mail services of the Eastern Bengal State Railway, which takes the traveller as far as Damookdeah, on the banks of the river Ganges. On arrival at Damookdeah the river is crossed by a large flat-bottomed paddle steamer. It has always been difficult to secure good landing jetties on this bank of the Ganges, owing to the shifting character of the sands forming the river banks. On the other side, but slightly above Damookdeah, is Sara Ghat, which is the terminus of the metre-gauge section of the railway, which serves the next link in the chain of travel.

The new bridge has been designed for a double broad-gauge track, and in order to guard against the deep scour of the river the foundations for the piers are to be carried down 150 ft. below the river bed. Each pier will be founded upon steel caissons, 63 ft. by 36 ft., which will be floated out to the required position and sunk, additional rings being put on and filled with concrete until the cutting edge of the caisson has reached the bed of the river. At the depth of 150 ft. a concrete plug will be placed in the bottom of the holes thus made, which will then be filled in with sand and a cap of concrete afterwards put on the top. All of the piers are to be constructed with a semi-circular nose and will extend to 60 ft. below low-water level.

The bridge will consist of fifteen spans, each 359 ft. in length and of heavy lattice girder type, having an overall depth at the centre of 52 ft. In addition to providing two broad-gauge railway lines there will be provision on the down-stream side for a footpath, carried on cantilevers on the outside of the main girders, 5 ft. in width, with light lattice parapet. The width, centre to centre of main girders, will be 32 ft., and the total width of the bridge, including the footway, 42 ft. 6 in. Each girder will be carried on massive steel bearings fixed at one end and movable at the other. The contract for these fifteen spans has been divided between the Cleveland Bridge and Engineering Company, Limited, Darlington, which is to manufacture nine, and Messrs. Braithwaite and Kirk, of West Bromwich, who are to supply the remaining six spans. Each span will have a weight of 1,300 tons, and shipment is to commence at the end of this year.

Severe competition was encountered from the leading American bridge companies, and the contract represents the most important individual steel bridge order placed for any part of the world in this country during recent years. The whole of the substructure work and the erection of the steel work is to be carried out by the Indian Government staff, under the personal direction of Mr. R. R. Gales, as engineer-in-chief, and according to designs prepared by Sir Alexander Rendel, acting as the consulting engineer for the India Office.

We hope to publish fuller details of this bridge, with illustrations, at a later date.

Port Talbot.—It has been decided to make considerable extensions in connection with the Port Talbot Railway and Dock Company, and the *Contract Journal* reports that this work will include a reinforced concrete wharf near the Port Talbot Steel Works.

New Loading Pier, Southend-on-Sea.—Southend-on-Sea T.C. has provisionally accepted tender of T. W. Pedrette, Enfield, amounting to £10,877 18s. 2d. (lowest received), for construction of proposed new loading pier in reinforced concrete.—*Municipal Journal.*

Woodbridge.—Bridge.—The Surrey C.C. have accepted the tender of Messrs. Somerville and Co., Victoria Street, London, at £4,800, for a new single-span reinforced concrete bridge at Woodbridge, Guildford.—*The Contract Journal.*

TRADE NOTICES, CATALOGUES, ETC.

A New and Improved Metal Lathing for Plasterwork.—We call special attention to the interesting announcement which appears in our advertisement pages, stating that the Expanded Metal Company, Limited, of York Mansion, York Street, Westminster, London, S.W., and Stranton Works, West Hartlepool, has succeeded in producing a new and improved diamond mesh expanded metal lathing which will be obtainable at prices lower than those for the original expanded metal lathings.

The new lathing will be known by the trade name of "BB Expanded Metal" lathing, and the illustration shows the actual size of its meshes. It is made in standard size sheets, 9 ft. longway of mesh by 2 ft. shortway of mesh.

It is stated to have several important advantages over the original expanded metal lathings. Its cost is stated to be less; then it may be used at wider spacings and it requires less plaster.

Several practical tests have been made, and the reports all agree that the new lathing is stronger and stiffer than the original lathings; and on account of the reduced size of the diamond mesh, together with the improved slope of its strands, less plaster goes through to form the "key" and practically none falls off and is wasted.

The new lathing is now ready for the market, and, as it is sold through the merchant trade only, stocks will be held in most large centres. It is listed in three gauges.

The price-list issued by the Expanded Metal Co., Ltd., argues that, gauge for gauge, a saving of from 25 per cent. to 60 per cent. is made by substituting the new lathings for the old. But the saving in cost is stated by the company to be really more, because of the wider spacings that may be used with the new lathings.

Tests have been made at these wider spacings with entirely satisfactory results.

A Well-planned Concrete Plant.—A plant lay-out which would be very suitable for buildings where about 10,000 cubic yards of concrete are to be placed is that which was used in building the American Can Company's new reinforced-concrete factory, located on Beach and Balmer Streets, Philadelphia. This building is five storeys high, with basement, and has over-all dimensions of 346 ft. by 100 ft. The $\frac{3}{4}$-yard Ransome mixer has turned out 9,200 yards of concrete for this work, at times mixing 290 yards in a trifle less than eleven hours, or 25 per cent. over normal rating. The plant has been so satisfactory that it has been taken down and is being used in constructing the new West Philadelphia High School, where about 8,000 yards of concrete will be mixed.

This plant is not only economical, but it is so simple that it can be taken down and re-erected with the greatest ease. The tower is of wood and 100 ft. high.

An inspection of the accompanying illustration will make perfectly clear the course the materials follow. The sand and stone, the latter mostly $\frac{3}{4}$ in., are dumped from wagons on to the platform and raised by bucket elevator, driven by a 10 h.p. engine, to the sand and stone bins at the top. From this point the aggregates fall by gravity into a proportioning hopper, being admitted through gates. The hopper is provided with a partition that can be set according to proportion desired.

Below the proportioning hopper is the fixed batch hopper of the $\frac{3}{4}$-yard Ransome mixer, operated by a 15 h.p. engine that runs the hoist also. The cement is piled in bags alongside of the top of the mixer's hopper, and water is turned into the latter from a pipe leading from a marked water-barrel connected to the city mains. The concrete is proportioned 1 : 2 : 4, and is discharged into a Ransome bucket hoist that is unloaded at the proper point. The contractors on this job are Cramp & Co., of Philadelphia.

ENQUIRIES.

July 21st, 1912.

SIR,—I was lately examining some paving slabs, the concrete of which was compounded of the ashes or clinker from a tramway generating station.

A very large percentage of this ash was unburnt, with the natural result that the slabs are useless. They are honeycombed with cracks, some $\frac{1}{8}$ in. in breadth,

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Illustration shows New Culvert 115 ft. × 5 ft. 4 in.
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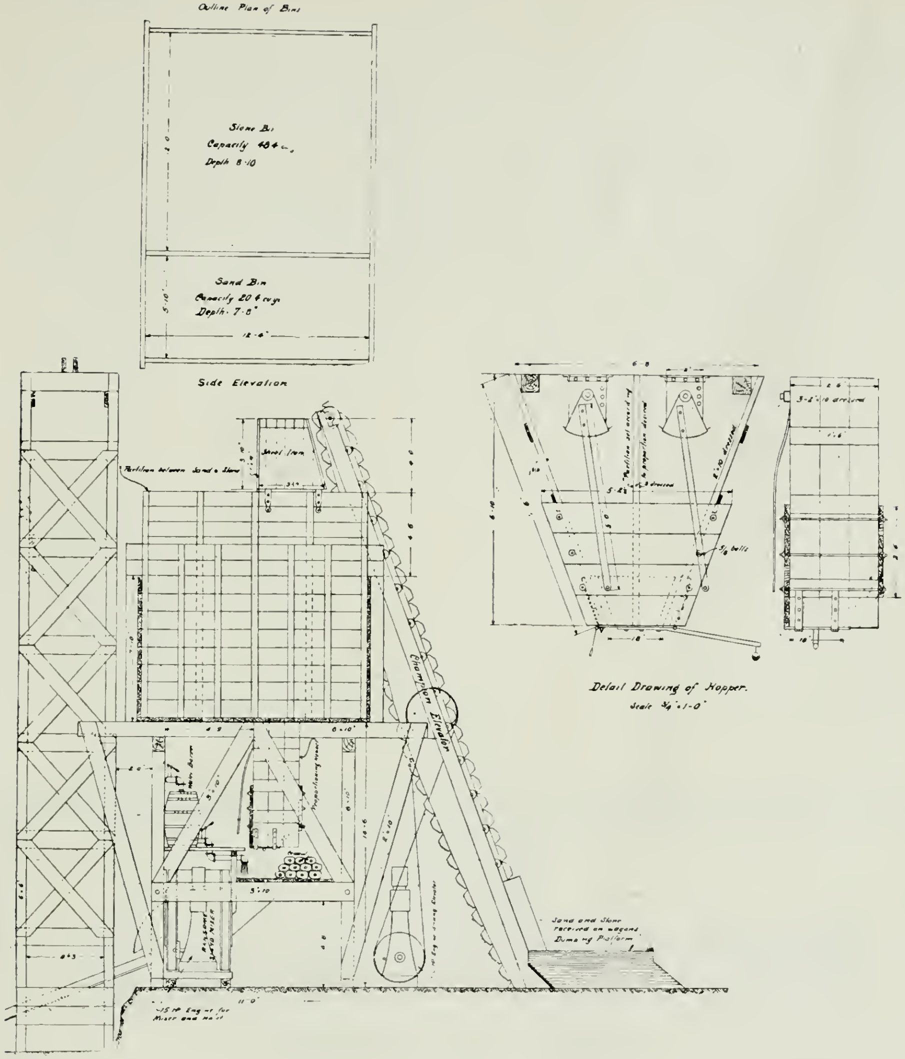

caused, I understand, by the lime in the cement acting on the sulphur or other substance contained in the unburnt material composing the concrete. I have been unable to find any particulars of the actual chemical action that takes place, and shall be greatly obliged if you can refer me to some mention of the subject.

Yours faithfully,

D. A.

The Editor, "Concrete and Constructional Engineering."

Reply.

Furnace clinker, if thoroughly burnt and free from dust, is undoubtedly a useful material where great strength is not required; it is very light, adheres well to the mortar, and resists great heat. Most coal, however, contains sulphur, generally in the form of iron pyrites, and part of this sulphur is likely to remain in the clinker as iron sulphide, a compound which is readily oxidisable in the presence of moisture. Should air and moisture gain entrance to the interior of the mass, a process which occurs with ease in so porous a material as clinker concrete, oxidisation takes place, accompanied by a considerable increase of volume, which may result in the complete disruption of the concrete. Instances are known in which fire-resisting floors have expanded from this cause, and have cracked and bulged with sufficient force to lift up and break an overlaying layer of asphalt. Experiments are needed to determine the permissible limit of sulphur, and should be undertaken by some institution like the Concrete Institute, and the conditions under which oxidisation takes place should also be investigated; in the meantime, it is advisable to regard furnace clinker as a material to be employed only with the utmost caution.

There is also the probability that the clinker may contain a great deal of unconsumed coal, which is not only a factor likely to cause disruption (more especially if the coal is of a bituminous character), but has also an adverse effect upon the physical character of the concrete, inasmuch as the coal easily breaks down under pressure. This is well illustrated by the following results of some tests made at the laboratory of the Associated Portland Cement Manufacturers, Ltd., two or three years ago:

Compression Resistance of Concrete Cubes	Lbs. per sq. in. 12 Months
Leighton Buzzard Sand 3 : 1	5950 5300 5625
Breeze 3 : 1	3000 3000 3000
Bituminous Coal 3 : 1	1200 1200 1200
Pit Sand 3 : 1	6300 6800 6550

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Breeze Partitions
Reinforced Sleepers
Telegraph Poles

Porous Pipes
Concrete Beams
Hollow Floors

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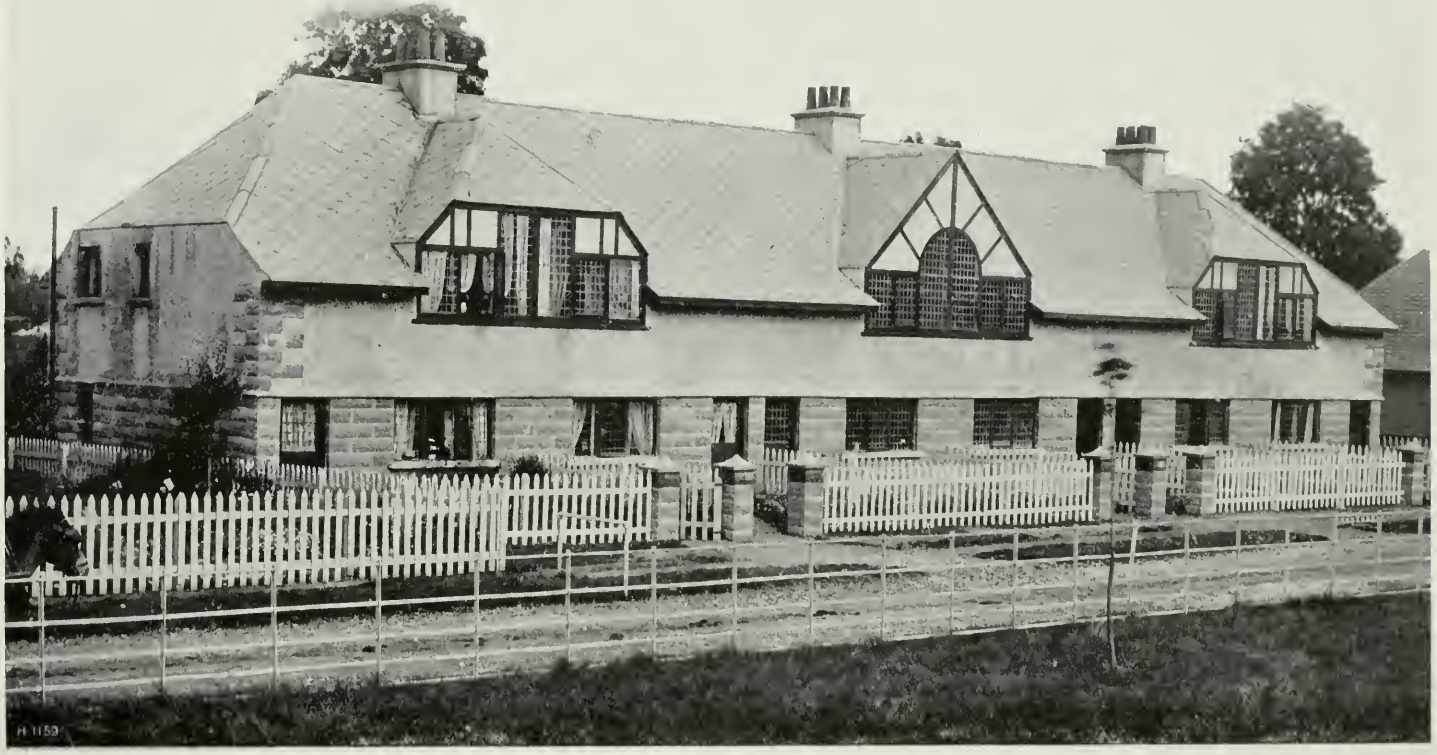


(For description see p. 732).
Made of "Winget" Blocks.
CONCRETE BLOCK COTTAGES AT TALBOT'S INCH, KILKENNY.

CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume VII. No. 10. LONDON, OCTOBER, 1912.

EDITORIAL NOTES.

TECHNICAL EDUCATION IN LONDON.

At this season we receive the curricula of quite a number of technical institutions where building construction is taught, but except in the case of one or two of the higher-grade establishments we fear the style of education accorded is more useful to the artisan, foreman, clerk of works, and the building superintendent than to the architect or to those who intend to make the building side of civil engineering their profession.

Though excellent work is done in the direction indicated, it would be well if the managers of such institutions were to realise the precise character of the education they offer and the class they cater for, and neither create illusion for themselves in this respect nor for those who provide the money for these institutions. When we see the London County Council School of Building, at Brixton, comparing its little structural engineering section—a most commendable department in itself—with a similar department at the great technical school of Charlottenburg, the former admirable institution invites ridicule, as, at its best, its teachings and its sphere of utility is that of a trade school or minor *Gewerbeschule* and not in any way on a par with that of a *Technische Hochschule*—*i.e.*, Imperial College of Science.

Apart, however, from this stricture as to the character of the teaching of the institutions who send us their curricula, we would naturally congratulate such an establishment as that of the School of Building at Brixton upon their foresight in according a certain amount of time and expenditure to a structural engineering section, so that the coming generation of foremen, clerks of works and building superintendents may be firmly grounded in the elements of these important subjects. That this Building School should also devote a considerable amount of time to questions of concrete and reinforced concrete is natural in this age, and we are glad that these subjects should be accorded the attention which they deserve.

We hope very much, however, that the character of the lessons and instruction given will not be beyond the capacity of the pupils—in other words, that the simplest of facts, so far as they are known, should be presented, and the simplest methods of practice be taught rather than what we should term the obtuse theories of the subject.

What we here say regarding the School of Building at Brixton may be also taken to refer to the Westminster Technical Institute, which is also under the *ægis* of the London County Council, and in which we find a course of reinforced concrete design and two in structural engineering provided for. Here, too, those who give the tuition should be very careful not to speak over the heads of their hearers.

Regarding the work of these two institutions to which we specially refer, we are presenting below that part of their programme which refers to the subjects with which this journal is connected :—

The L.C.C. School of Building, Brixton.—A Senior Day Technical School has been established to provide instruction for students whose age and previous education are above the age and standard of education required for admission to the Day Technical School for Boys. The instruction is arranged in the following sections: (1) General Building; (2) Architecture; (3) Surveying; (4) Structural Engineering. The curriculum will include instruction in Building Construction, Architectural Drawing, Reinforced Concrete and Structural Engineering, Mechanics of Building, Chemistry and Physics of Building Materials, and practical training in Brickwork, Masonry, etc.

The lectures on drawing and design of the Constructional Steelwork of buildings are given by Mr. A. R. Sage, assisted by Mr. F. C. Webster, A.R.I.B.A. The lecturer on Mechanics of Building and Strength of Materials is Mr. A. R. Sage, assisted by Mr. E. G. Beck, A.M.Inst.C.E., and Mr. S. J. Oxer. Reinforced Concrete and Structural Engineering as Applied to Building are taught by Mr. H. Kempton Dyson, the Secretary of the Concrete Institute, assisted by Mr. R. Graham Keevil, A.M.I.Mech.E., and Mr. A. E. Crabbe. The lectures on reinforced concrete are amplified by experiments, diagrams, lantern slides, specimens, models, and the use of testing plant. An endeavour will be made to visit works in course of construction so as to acquaint students with actual working conditions.

Westminster Technical Institute.— In this Institute there are two sections devoted to structural engineering, the lectures being given by Mr. E. H. Sprague. Structural Design in Reinforced Concrete is taught by Mr. E. S. Andrews, B.Sc., on Wednesdays from 7.15 to 9.30 p.m. This is an advanced course of fifteen lectures with practice in design, for civil engineers and others already well grounded in the theory of structural design. The syllabus is as follows :—The properties and tests of concrete; the theory of reinforced beams; methods of calculation with examples of simple and doubly reinforced beams, floors and T beams, simply supported and continuous; calculations of stirrups; columns axially and eccentrically loaded; calculations for a concrete arch, reservoirs, retaining walls, etc.

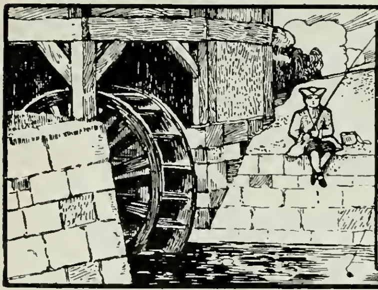
CHEAP CONCRETE BLOCK COTTAGES.

We would call special attention to some illustrations in this issue, which indicate the use of concrete blocks and roofing materials in cottage work.

In our Frontispiece is shown some artistic cottages at Talbot's Inch, Kilkenny, and on page 732 there is an article dealing with concrete agricultural cottages in Norfolk.

One of the arguments against the use of concrete is an absence of artistic appearance, but, although we in no way wish to say that the artistic merits of the buildings illustrated could not be materially improved upon without additional expense, still, in the hands of good designers, artistic effect could be obtained at a small outlay of money.

We have from time to time referred to the uses of concrete for cottage building, and would again urge those concerned to consider carefully the advantages of this material. There is no doubt that in many districts concrete is eminently more economical than stone or brick. Its utility has by no means been thoroughly recognised as yet, due perhaps to the fact that the advent of concrete for building purposes has been much overshadowed by the production of reinforced concrete systems. But, as a matter of fact, the field for the concrete block is even greater than for reinforced concrete. Its economical advantages are equal, if not greater; and we would, therefore, again call attention to its use for cottages, and suggest that the subject should receive careful attention from those who have economic problems of housing before them.

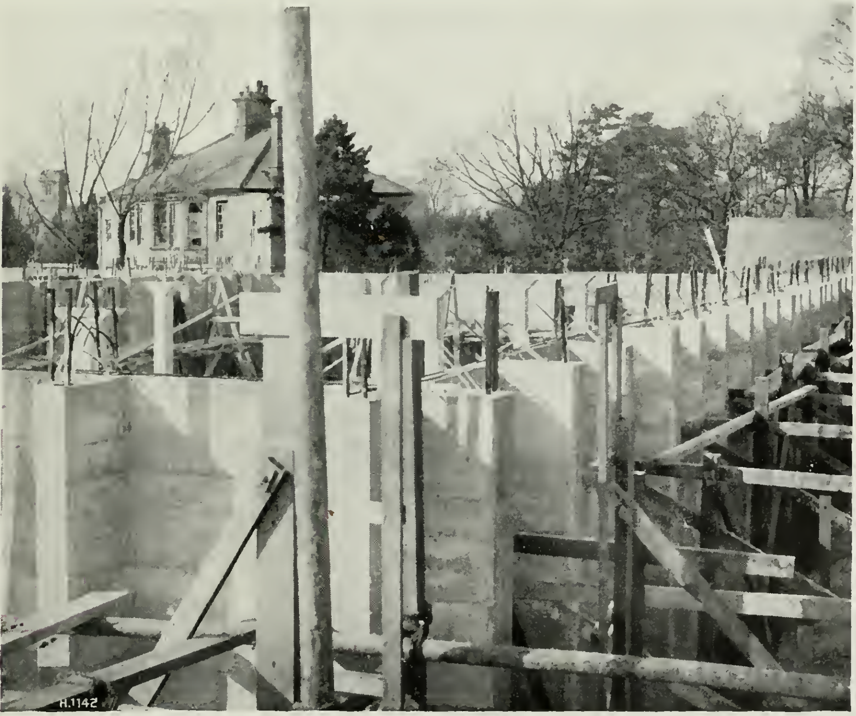


REINFORCED CONCRETE RESERVOIR FOR THE EAST COWES URBAN DISTRICT COUNCIL.

Below will be found a description of a large Reinforced Concrete Reservoir at East Cowes, where this method of construction was adopted owing to the great economy to be effected by its use, and we commend the perusal of the article to all public authorities concerned with the erection of reservoirs and other water works.—ED.

THE above Council have just completed the construction of a reservoir which will be in all probability extensively duplicated by authorities who adopt scientific methods of construction.

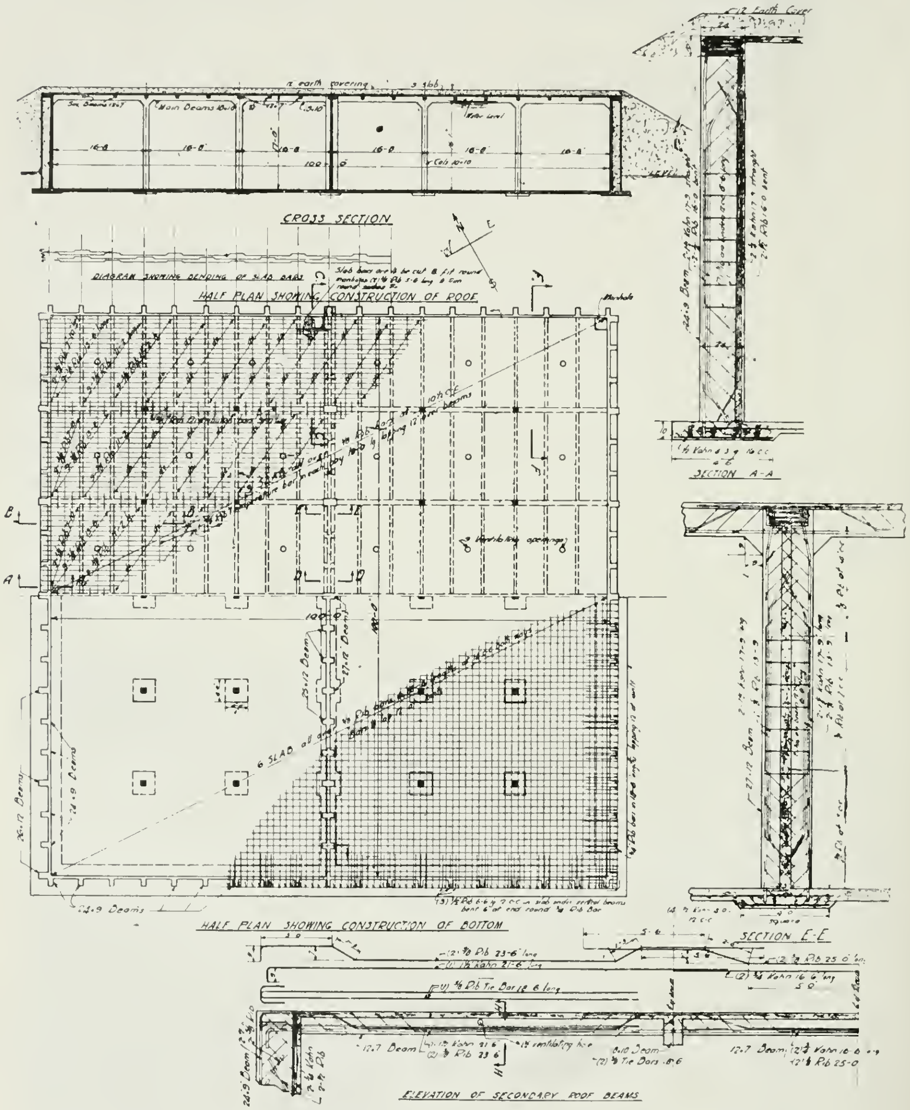
Some months ago the Council instructed their Water Engineer and Surveyor (Mr. Albert E. Barton, M.I.Mun.E.) to prepare plans for a brick reservoir capable of holding 1,000,000 gallons. Upon the completion of these plans it was found that the cost was prohibitive, and on the recommendation of Mr. Barton the Council decided to construct it in reinforced concrete, the result of which was that the cost worked out at considerably less than half the cost of the brick-built reservoir.



External View showing Buttresses.
REINFORCED CONCRETE RESERVOIR FOR THE EAST COWES URBAN DISTRICT COUNCIL.

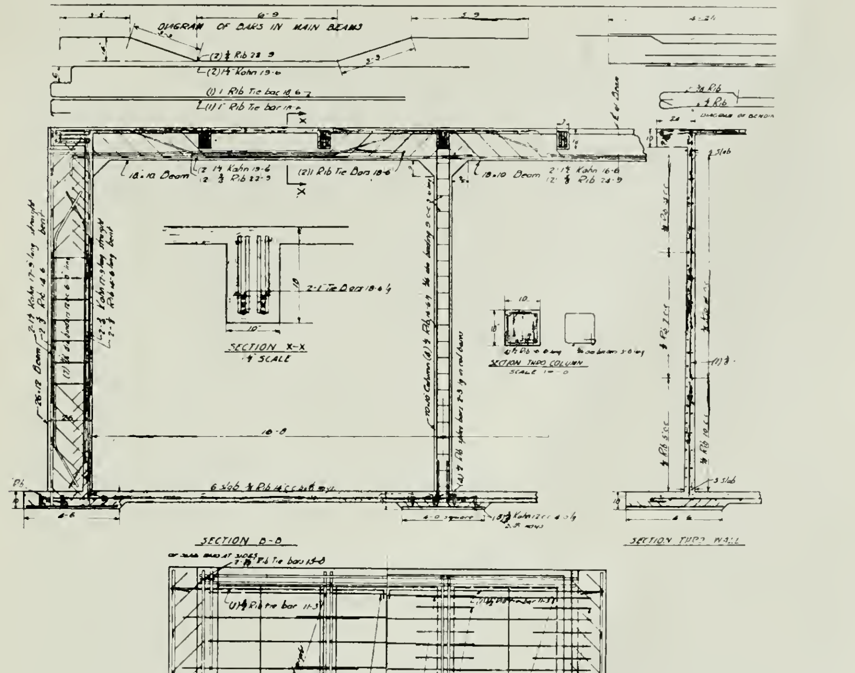
Tenders were advertised for, several well-known reinforced concrete engineering firms submitted designs, and the Council, after careful consideration, adopted the "Kahn" Trussed Bar System, submitted by the Trussed Concrete Steel Co., Ltd., Caxton House, Westminster.

The internal dimensions of the reservoir are 100 ft. by 100 ft., with a clear height of 17 ft. between floor and roof slab; the overflow provides for a maximum depth of 16 ft., the total capacity being 1,000,000 gallons. The reservoir is divided into two parts, having a division wall in the centre, which can be worked independently of each other.


CROSS SECTION
DIAGRAM SHOWING BENDING OF SLAB BARS
HALF PLAN SHOWING CONSTRUCTION OF ROOF
HALF PLAN SHOWING CONSTRUCTION OF BOTTOM
SECTION A-A
SECTION E-E
ELEVATION OF SECONDARY ROOF BEAMS
Manhole
Ventilating openings
16-0
100-0
A
B
E
24.9 Beams
27.12 Beam
12.7 Beam
8.10 Beam

Half Plan and Sections.
REINFORCED CONCRETE RESERVOIR FOR THE EAST COWES URBAN DISTRICT COUNCIL.

The site of the works is a few yards distant from the Royal Naval College, Osborne, on the main road between East Cowes and Newport, with a surface level of 173 ft. above O.D., subsoil stiff, loamy gravel. The top soil to a depth of nearly 5 ft. was excavated for the reception of the foundations and floor slabs, the materials excavated being used to form bankings and coverings to the portions above ground.



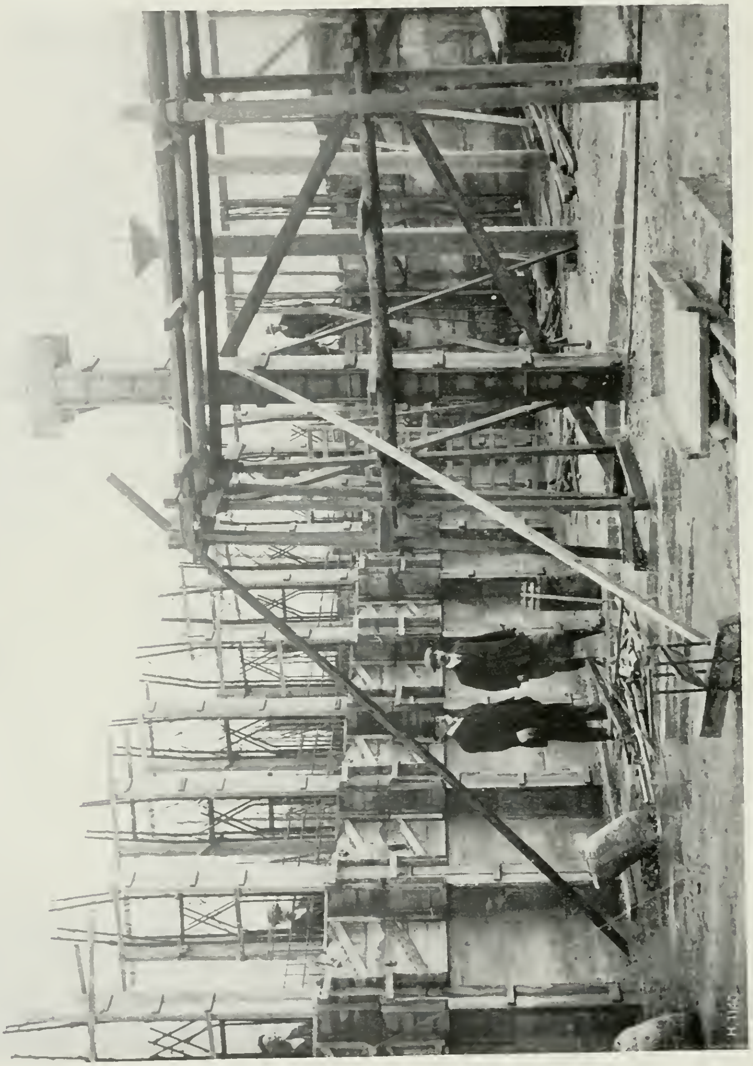
Sections.

Reinforced Concrete Reservoir for the East Cowes Urban District Council.

The floor slab consists of 6 in. concrete interlaced with ⅜-in. rib bars, spaced in both directions at 14-in. centres.

The wall slabs between vertical buttresses or piers have horizontal reinforcing bars spaced 4 in. to 12 in. apart at varying heights from the bottom proportionate to the pressure; the vertical bars are placed in the centre of the bay, the panels and piers are flush on the inner side. The centre division wall has the panel in the centre of piers; the thickness of the slab is from 5 in. at the floor level to 4 in. at the top.

The roof slab is 3 in. thick, with ⅜-in. cup bar, 10½-in. centres, the maximum size of slab clear of beams being 5 ft. by 16 ft. 8 in.; 12 in. of surface soil is placed on the top, with the usual ventilators for each bay.



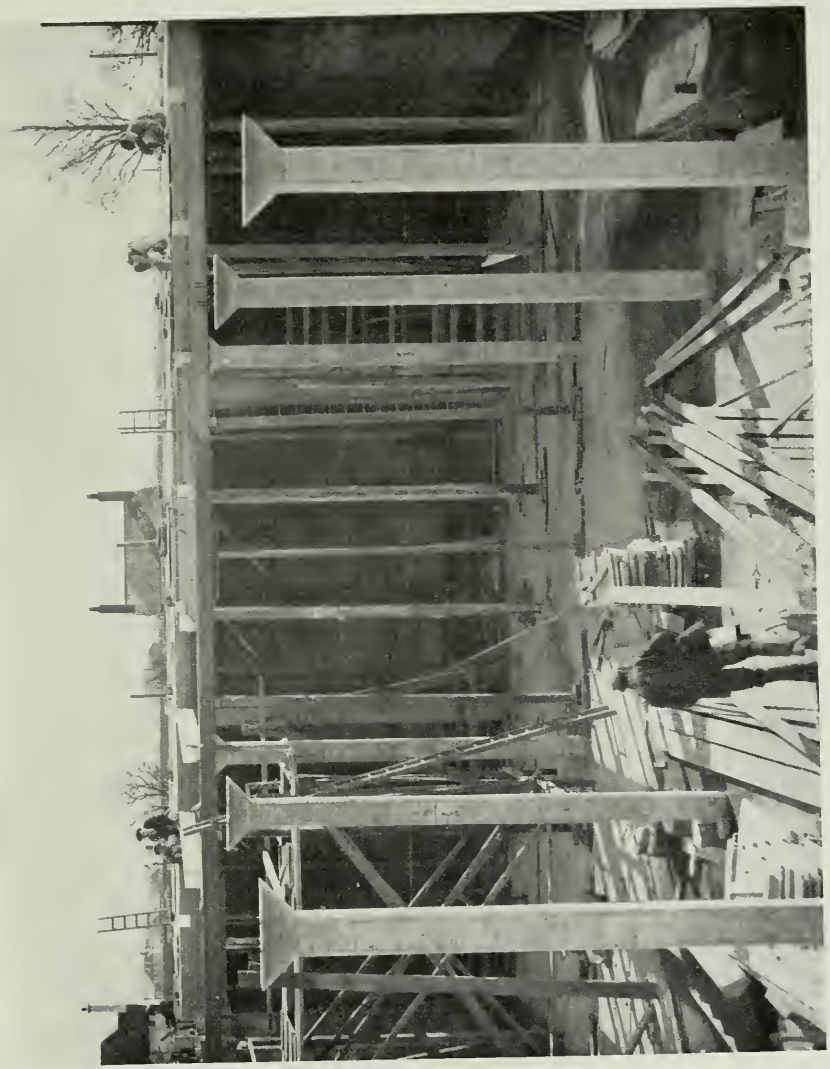
Centre Wall showing Shuttering to Columns.

Reinforced Concrete Reservoir for the East Cowes Urban District Council.

The east and west outer walls consist of seven 26 in. by 12 in. vertical buttresses reinforced with "Kahn" trussed and cup bars, opposite the cross line of columns, and twelve 24-in. by 9-in. intermediate buttresses, all standing upon a continuous beam 24 in. by 10 in., into which the roof beam and slab tie bars are hooked.

The remaining outer walls are constructed in a similar manner with nineteen 24-in. by 9-in. buttresses and 4-in. to 5-in. wall slabs between; no cross beam at head is required.

The dividing wall is formed of five 27-in. by 12-in. and twelve 25-in. by 12-in. vertical buttresses of similar construction to last, with 5-in. to 4-in. slab spanning between these buttresses.

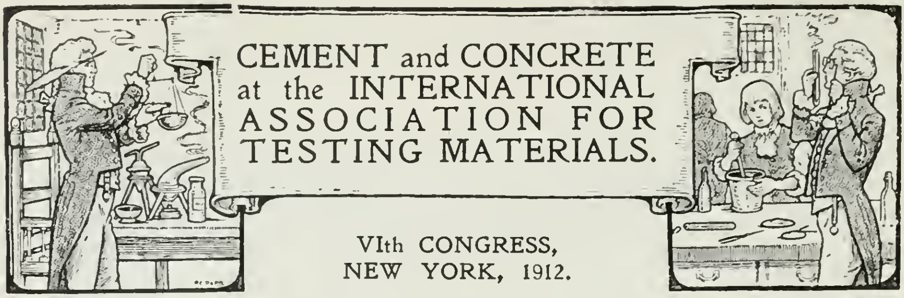


Reinforced Concrete Reservoir for the East Cowes Urban District Council.

The materials used consist of "Needles" shingle and sea sand mixed with best Portland cement, from the Newport, I.W., Branch of the Associated Portland Cement Co.'s manufacture, the principal portions being five parts shingle ($\frac{3}{4}$ mesh) and clean, sharp, sea sand to one part of Portland cement.

The internal walls are coated with 1 in. of cement rendering gauged three sand and one cement, and mixed with "Ceresit" waterproofing composition; the rendering is done in three coats and finished with plain floated surface.

Messrs. W. H. Brading & Son, East Cowes, I.W., were the contractors.



CEMENT and CONCRETE at the INTERNATIONAL ASSOCIATION FOR TESTING MATERIALS.

VIth CONGRESS,
NEW YORK, 1912.

We are publishing in this issue some further papers which were presented at the recent Congress in New York of the International Association for Testing Materials.—ED.

THE BOILING TEST AND THE VOLUME CONSTANCY OF PORTLAND CEMENT IN DRY STORAGE.

By Professor F. SCHÜLE, Zürich.

THE Swiss standard specifications for testing hydraulic cements demand, in addition to the cold-water test, accelerated tests for the constancy of volume. In the case of Portland cement the hot-water test at 100° C affords a characteristic criterion whenever the cement is to be used in a dry atmosphere, which is the usual case with reinforced concrete superstructures.

In his fundamental investigations of hydraulic cements L. von Tetmajer left nothing undone to determine the value of accelerated tests, especially of the boiling test. We owe it to him that, since the eighties, cement briquettes have been preserved in Zürich, after setting in damp air, for years under water and in dry air, and that periodical revisions have been held in order to determine the reliability of the accelerated tests for constancy of volume. Up till 1892 he has himself published the results of those revisions. The subsequent re-examinations have likewise confirmed the reliability of the boiling test as an accelerated method for determining the constancy of volume of Portland cement when exposed to the dry atmosphere.

Investigations.—In the period 1893 to 1902, 2,200 Portland cements have been examined at Zürich. Of these 9 were found blowing and had developed expansion cracks after 28 days under water, 193 cements had not stood the boiling test, and of these :

145 were soft and exhibited many reticulated cracks;
48 had decayed.

After being kept in dry air the re-examination up to 1908 showed that

100 had turned blowing and soft;
29 had decayed.

A total of 129—that is, 67 per cent.—of the cements which had not stood the boiling test.

In the interval 1903 to 1908, 1,737 Portland cements have been examined. The subjoined table gives the results of the boiling test, the cold-water test, and of the revision of 1910 for the separate years.

We see that 84 cements have not stood the boiling test. Of these 57 exhibited reticulated cracks and were soft, and 27 had decayed—that is, 4·9 per cent. of the total.

The revision of 1910 proved 25 cements to be doubtful, 10 had decayed; thus 35 had deteriorated by exposure to air, which means 41·7 per cent. of the cements which had not stood the boiling test. Of the other cements none had decayed, eight were doubtful, one was soft and reticulated—a total of nine, or 0·52 per cent. of those which had stood the boiling test.

The importance of the boiling test is clearly brought out by these figures.

The inferior cements were not all products of faulty composition, which would be revealed by low strength and by blowing during the cold-water test; there were also some products of standard composition, and in their case the cause of the decay in air should be inquired into.

Under water, hardening results by the binding of an amount of water, which increases with time. After one year the chemical analysis, if we take the means of three good Portland cements, will show:

11·51 per cent. of bound water;
3·35 per cent. of carbonic acid.

In the air the same cements will yield the following results:

	after 3 days.	7 days.	28 days.	1 year.
bound water, per cent.	6·29	7·17	8·09	5·97
carbonic acid, per cent.	3·50	3·49	3·95	7·71

These changes continue. Faultless cements will, according to Tetmajer, show the following averages of 15 analyses after several years:—

RESULTS OF THE REVISION OF 1910, VOLUME CONSTANCY OF PORTLAND CEMENTS 1903 TO 1908.

Year of Examination	Number of Cements Tested	A Behaviour in the hot water Test at 100° C			Found blowing in cold water at Revision	Of the Cements A kept in dry air there were found in 1910				Cements which stood the boiling Test, but were found, after being kept in dry air, in 1910			
		Reticulated, soft	Decayed	Total		Doubtful	Reticulated, soft	Decayed	Total	Doubtful	Reticulated, soft	Decayed	Total
1903	278	7	3	10	0	4	0	1	5	1	1	0	2
1904	284	5	2	7	0	1	0	1	2	2	0	0	2
1905	290	3	3	6	2 already after 28 days	2	0	0	2	0	0	0	0
1906	248	9	2	11	0	6	0	1	7	0	0	0	0
1907	319	18	13	31	1 already after 28 days	8	0	5	13	3	0	0	3
1908	318	15	4	19	4 already after 28 days	4	0	2	6	2	0	0	2
Total	1737	57	27	84	7	25	0	10	35	8	1	0	9
%	—	3·3	1·6	4·9	0·4	29·8	0	11·9	41·7	0·46	0·06	0	0·52

Eidgen. Materialprüfungsanstalt Zürich.

	Bound H_2O	CO_2
stored under water	10·44 per cent.	2·61 per cent.
stored in the air	2·95 per cent.	12·77 per cent.

whilst blowing cements will yield (averages of 15 or 20 analyses respectively):

	Bound H_2O	CO_2
stored under water	11·19 per cent.	1·60 per cent.
stored in the air	3·59 per cent.	9·88 per cent.

An essential difference in the behaviour of the two classes of cements cannot be recognised. In the atmosphere the percentage of bound water diminishes gradually, while the percentage of the carbonic acid, which combines with the liberated lime, is increasing. The strength of the material changes in accordance. Plastic samples of a 1 : 5 mortar showed, after two years and a half, when hardened in the air and under water, the ratios:

0·61 for the bending strength,
0·54 for the compression strength.

The Portland cement used in these experiments was found sound in the boiling test. We may imagine that these ratios will become very small in the case of mortars and concretes made from cements which had not stood the boiling test, nor had been able to withstand the chemical reactions which are to be expected in such cases.

The boiling test has apparently little importance for buildings and structures which are in the open, or under water, or exposed to a damp atmosphere. For parts of buildings, however, which are to remain in dry air, the boiling test will afford valuable information as to the reliability of the cement.

Cements which turn soft during the boiling test or which decay should be excluded from the latter kind of buildings, though detrimental influences may not become apparent within the first years. In the opinion of the author the whole future of reinforced concrete superstructures is dependent upon the use of cements such as will not undergo a slow destruction by exposure to air. The accelerated boiling test at 100° C supplies us with a reliable means of eliminating doubtful cements.

TESTS FOR CONCRETE.

By CLOYD M. CHAPMAN, New York.

Testing Engineer for the firm of Westinghouse, Church, Kerr & Co.

The art and the science of using concrete have developed so rapidly during the past score of years, and to-day this material is used for so many purposes and under such a multitude of conditions by operators having such varying degrees of knowledge of the work, that one cannot but question the adequacy of the present supervision of the concrete produced.

With all the wonderful growth of the industry and its rapid advance into new fields, there is no recognised standard test or specification now in use for concrete.

The prescribed standard laboratory tests for the cement and the aggregate may be carefully and accurately carried out, and may show the materials to be good for the purpose, yet if the workmanship is not equal to the materials the result will be disappointing. A test of the concrete after it is made is the only means of knowing the quality of the work done.

It is hardly possible that a single test or specification should be applicable to all concrete, but it would seem that it is indeed time that steps be undertaken to develop a standard test for concrete in foundations, bridges, dams, buildings, and similar work.

It is the purpose of this paper to enumerate some of the various forms of test which might be applied to such concrete, in the hope that the ensuing discussion may lead to further action in the direction of securing the development and adoption of some form of test which will show the quality of the concrete which enters into such important structures.

This paper does not advocate the adoption of any particular one of the tests mentioned. Some of them are not worthy of serious consideration. But a standard test is needed very urgently, and from among the following forms of tests there should be developed a test which could be relied upon to give a fairly accurate indication of the quality of the concrete in a finished piece of construction.

Moulded Specimen Compression Test.—A method of test which may be called the "Moulded Specimen Compression Test" is already used to some extent, although there seems to be little uniformity in the details of the test as carried out by various engineers. It consists in making in the field, at the site of the work, from concrete which is being placed in the forms, one or more specimen cubes, prisms, or cylinders, which are subsequently crushed in a compression machine.

The form and size of these test specimens vary greatly. Cubes from 2 in. to 12 in. on a side, and cylinders from 6 in. to 12 in. in diameter are used, while the length of specimens varies from 2 in. to 18 in. The moulds used vary from rough wood boxes to accurately machined metal forms. When rough wood moulds are used they frequently perform the additional service of packing-boxes in which the specimen is shipped to the laboratory for test.

Among the advantages of this form of test are :—The ease with which the specimens are prepared in the field, and the accuracy with which the strength of the specimen may be determined in the laboratory. Among its disadvantages are :—The concrete tested is not the concrete which is in the structure, the specimen set and hardened under conditions not identical with those of the structure, and the specimen may not truly represent the quality of the concrete in the remainder of the structure.

In order that the results obtained from tests made by any method may be more uniform, and that results obtained by one party may be comparable directly with those obtained by others, it is necessary that methods and conditions shall be standard as regards those matters which affect the results. To this end it would be necessary to define the following details for this form of test : (1) The form of the specimen ; (2) the size of the specimen ; (3) the method of moulding the specimen ; (4) the method of storing the specimen until tested ; (5) the age at which the specimen shall be tested ; and (6) the method of testing the specimen.

Moulded Specimen Tension Test.—A second method which may be used is a modification of the one just described, and differs from it in that the specimen is tested in tension instead of compression. In general this method has the same advantages and disadvantages as the first-mentioned test, except that there is liable to be greater difficulty in securing an even distribution of stresses throughout the cross-section of the specimen in the tension test than in the compression test. The details of the test which would require standardising are practically the same as those for the compression test.

Cut Specimen Test.—A third method is one which has also been to some extent, particularly on old concrete, or on concrete taken from structures which have failed or been subjected to extreme conditions, such as fires, flood, etc. This method may be called the "Cut Specimen Test," and consists in cutting from the mass of concrete a specimen in the form of a cube, prism, or cylinder (by means of core drill or otherwise), which is afterwards crushed in a compression machine.

The size and form of the specimens taken for this test also vary within wide limits, but the specimens are often rather small, because of the difficulty of cutting.

Among the advantages of this method are :—The specimen tested is a part of the structure ; the specimen set and hardened under the same conditions as the remainder of the structure ; and any portion of the completed structure may be tested, unless reinforcement interferes. Among its disadvantages are :—The difficulty in cutting out a suitable specimen ; the objection to cutting into new work and patching up the hole left ; the impossibility of cutting out a suitable specimen from certain very important parts of some structures on account of the network of reinforcement ; and the possibility of weakening the specimen when it is removed by drilling around it.

The details of this test which it would be necessary to standardise to make results comparable are : (1) The size of the specimen ; (2) the form of the specimen ; (3) the method of removing the specimen from the structure ; (4) the age at which the specimen shall be removed ; (5) the method of storing the specimen between the time of removal from the structure and testing ; (6) the age at which the specimens shall be tested ; and (7) the method of testing the specimen.

Penetration Test.—A fourth method may be called the "Penetration Test." In its crudest form it consists in attempting to drive a nail of a given size into the concrete at a fixed time after moulding, noting whether the nail can be driven in or whether it bends before penetrating to its full length.

This test has had very little application. Its chief advantage is its simplicity, and its greatest drawback is its total lack of accuracy. Little, if any, consideration need be given this test in the form described above.

Indentation Test.—A modification of the penetration test which might be applied to concrete is similar to that employed to compare the hardness of metals. It may properly be called the "Indentation Test." It consists in driving a hard body of some more or less pointed form into the surface of the material under test by means of impact or pressure and measuring accurately the penetration of the body into the surface being tested.

This test has not been used for concrete, so far as we know, and the value of the results to be obtained is therefore problematical. Among its advantages are:—The concrete tested is a part of the structure; it has set and hardened under identical conditions with the remainder of the structure; any portion of the surface of the structure which is accessible may be tested; the same concrete may be tested at different ages as it hardens; simplicity of the apparatus and the test. Among its drawbacks are:—The concrete is tested at restricted spots on the surface, and not in a mass; doubt as to proper meaning of the results obtained until long experience has taught how to interpret them; the heterogeneous character of concrete making it necessary to test in many places to gain a fair average; and only the surface concrete is tested.

For this test the following details should be standardised before results could be comparable:—(1) The exact form and size of the penetrating body; (2) the amount of pressure or size, and number of impacts to be used in driving; (3) the method of applying the pressure or blows; (4) the method of measuring the penetration.

Duplicate Member Test.—A method which has been used in some few instances on very important work may be called the "Duplicate Member Test." It consists in moulding, at the same time as the main structure, a duplicate (including reinforcement), or a portion such as a beam, girder, slab, or arch, and loading this specimen to its breaking point after it has attained a certain age. This method is sometimes carried out on models of members something less than their full size in the structure.

Some of its advantages are:—The concrete is tested under conditions of stress similar to those in the structure; it serves as a check on the adequacy of the design; it develops the weak spot by its manner of failure; and the concrete tested, if proper care is taken, may set and harden under very nearly the same conditions as the structure. Its chief disadvantages are:—The concrete tested is not the concrete in the permanent structure; the concrete tested, if proper care is not taken, may not set and harden under the same conditions as the structure; and the test is expensive of time, labour, and material.

To compare results obtained by this test the following details would have to be standardised:—(1) The method of preparing the test member; (2) the method of storing the test member; (3) the method of testing the test member; and (4) the method of interpreting the results.

Standard Member Test.—Another form of test which has been used, a modification of the duplicate member test, consists in moulding and testing a standard beam having standard reinforcement, which beam and reinforcement are not duplicates or models of any beam in the structure. This method may be called the "Standard Member Test." It has been advocated and used by Von Emperger, of Vienna. In this test beams of uniform size and shape, reinforced in a uniform manner, are made up with the concrete being used on the work. After ageing, the beams are loaded to failure, and from the data obtained the strength of the concrete is computed by means of established formula.

The chief advantages of this form of test are:—The concrete is tested under conditions of stress very similar to those it is called on to resist in a reinforced structure; with proper care the concrete may set and harden under conditions very similar to those existing in the structure; and the apparatus required and the test are comparatively simple. Some of the disadvantages of this test are:—The concrete tested is not the concrete in the structure; and unless proper care is taken the concrete may set and harden under very different conditions from those existing in the structure.

The following details require standardisation:—(1) The form and size of the test beam; (2) the method and details of reinforcement; (3) the method of moulding the beam; (4) the method of storing; (5) the age of breaking; (6) the method of loading; and (7) the interpretation of results.

Tensile Test.—An eighth form of test for concrete in place has been used in at least one case in this country. It may be called "Tensile Test," and consists in embedding in the concrete when it is placed in the forms pieces of metal of deformed shapes, which are afterwards pulled out of the surface in such a manner that the force required to extract the metal is measured.

This test has been carried out by embedding a few inches of the end of a deformed reinforcing rod in the concrete and after a certain period of time pulling out the bar with a lever. The crudeness of this form of test is so evident that no further consideration will be given it at this time. It serves, however, to suggest more practical forms of tests, and is mentioned here only for that reason.

Improved Tensile Test.—An improvement over this method consists in using in place of pieces of deformed reinforcing bars a metal body of such design that in withdrawing it there would be no possibility of the bond between metal and concrete yielding in place of the concrete itself.

The chief advantages of this form of test are:—The concrete tested is a part of the structure; it has set and hardened under the same conditions as the structure; the test may be applied to almost any accessible predetermined part of the surface of the structure; and the test is simple and easy to perform. Among its disadvantages are:—Only previously selected points which have been prepared while the concrete was being placed can be tested; there will be doubt as to the proper interpretation of the results obtained until after long experience in the use of the test; and the difficulty of measuring accurately the area of the cup-shaped fracture.

For this test the following details would have to be standardised:—(1) The form and size of the metal body; (2) the depth to which it is embedded; (3) the method of embedding; (4) the method of drawing the body; and (5) the method of interpreting results.

Modified Tensile Test.—A slight modification of this "Tensile Test" will overcome some of these objections, and constitutes a tenth form. The modified test consists in drilling a hole in the surface of the concrete which is larger at its bottom than at the entrance. Into this hole is securely bedded a metal plug, which is then extracted as in the Tensile Test.

The chief advantages of this form of test are:—The concrete tested is a part of the structure; the concrete tested has set and hardened under the same conditions as the structure; any accessible portion of the surface of the structure may be tested; and the test is simple and easy. Among its disadvantages are:—Doubt as to the proper interpretation of results, and the difficulty of measuring the area of the cup-shaped fracture.

For this test the following details require standardising:—(1) The method of drilling the hole; (2) the method of embedding the metal body; (3) the form and size of the metal body; (4) the method of withdrawing the body; and (5) the method of interpreting the results.

Other Tests. — There are still other methods which might give indications of the quality of finished concrete:

An "Absorption Test," indicating the density of the mass; and a "Deformation Test," in which the deformation of a member in a structure under known loads is accurately measured with delicate instruments.

Even these twelve methods enumerated probably do not cover the entire available material at our disposal. Such is not the purpose of this paper. Its principal aim is to point out that there is a variety of tests which might be used, and to urge the necessity of the adoption of some method or other as a standard.

The writer does not advocate any particular method, nor does he believe that any one of those mentioned is perfect. Yet it is so important that we have some kind of a test for finished work which shall be accepted as a standard that it seems the duty of this Congress to take some definite step towards that end.

ACCELERATED TESTS FOR CONSTANCY OF VOLUME OF CEMENT.

Report on the Principal Question by BERTRAM BLOUNT, London.

Since the time when the use of accelerated tests for determining the constancy of volume of cement became common, a very large number of experiments have been made by many different operators, and have shown that, although it is possible that some cement may fail to comply with such tests and yet be stable under working conditions, yet no case has arisen where a cement which has complied with these tests has failed under working conditions from want of constancy of volume.

I conclude that this mode of testing is of high utility and that its omission from any code of testing renders that code imperfect and obsolete. Support for this belief is afforded by the fact that in this country manufacturers and users alike have accepted it generally, and have used it regularly as a means of control to their joint satisfaction, both parties recognising that by this means they avoid many causes of dispute as to quality, and acquire a valuable safeguard against latent defects which in the course of time may lead to serious structural disasters.

In order to show that the growth of understanding between the manufacturer and the user has been progressive, I have tabulated some figures obtained during the last eight years, by the Le Chatelier test, and append them to this report. It will be seen that since the British Standard Specification was officially promulgated in this country the requirements as to constancy of volume have been made more stringent, and at the present time the maximum permissible expansion of cement tested by the Le Chatelier method is fixed at 10·0 mm. There has been no difficulty in obtaining cement complying easily with this requirement, and many manufacturers habitually produce a cement the expansion of which falls well below this limit.

The idea that the cement made in other countries falls short of the quality of that produced in the United Kingdom would not be entertained by any responsible person, and the conclusion that such cement is capable of passing a rigorous accelerated test for constancy of volume is consequently irresistible.

Having regard to the substantial practical advantages which the use of a simple and reliable test of this class has been found to possess during a period of probation extending over years, in the course of which it has aided in controlling the acceptance and employment of a good many million tons of cement, I am of opinion that the international adoption and general use of such a test will prove to be of the highest value to a great and growing industry, and should receive the support of all concerned with that industry, whether as manufacturers, users, or technologists.

Le Chatelier Tests on samples of Portland Cement.
Summary.

Specification.	Maximum 12·0 mm.					Maximum 10·0 mm.					
	1903	1904	1905	1906	1907	1907		1908	1909	1910	1911
					To June 30th	June to Decr.	Calculated on old specification				
Total Number of Tests	73	163	169	158	121	114		127	105	153	93
Mean Result mm	15·1	10·5	7·8	6·7	6·8	5·7		4·6	4·2	2·4	3·0
Highest Result mm	122·0	54·0	51·0	51·0	50·0	18·5		47·0	40·0	24·5	30·0
Lowest Result mm	0·5	0·5	1·0	0·0	0·5	0·0		0·5	0·5	0·5	0·0
No. of Samples above Specification	23*	49	30	25	19	27	15	12	8	1	3
Percentage of Samples above Specification	31·5	30	17·7	15·8	15·7	23·7	13·2	9·4	7·6	0·7	3·2

* Assuming 1904 British Standard Specification requirements, viz., 12·0 mm. Maximum.

ON THE TESTING OF HYDRAULIC BINDING MEDIA.

By J. BIED, Viviers (Ardèche).

I.

During the last twenty years or so, hot water tests have been currently applied to hydraulic binding media in France, and the employment of the Le Chatelier needle cylinder apparatus for this purpose has been absolutely general and free from any objection.

II.

Having read with a fair amount of attention the publications on this subject, I think that, leaving out of consideration the question of knowing whether or not the expansion in cold water previous to immersion in hot water should be disregarded—which point is quite independent of the test itself—the objections urged against the Le Chatelier hot test can be reduced to three:

(*a*) The test would lead to the rejection of cements which would have behaved well in cold water;

(*b*) if the tests be applied immediately after grinding, it will lead to the passing of cements which would, on the contrary, be rejected if the test were not applied until after aeration for a fortnight or a month;

(*c*) finally, the test lacks precision, and furnishes results deficient in concordance, when employed in the same or in different laboratories.

These three objections will now be examined in turn.

III.

The Le Chatelier test is likely to cause the rejection of good cements.

Here it is at once necessary to state the case properly, which does not seem to have been done.

From a general point of view, the reproach is perhaps well founded, for I have kept in my laboratory for several months specimens of cements which gave an expansion of 50 mm. under the Le Chatelier test without exhibiting any trace of expansion in cold water.

If, however, one takes up the point of view of the consumer, the reproach is unfounded, for the two following reasons:

(*a*) The Le Chatelier test seems to eliminate all bad cements; and this is the main thing.

(*b*) Any manufacturer who knows his business can, without any sensible addition to the cost of production, make cements that will pass the Le Chatelier test.

Though it is not my place to point out what means should be adopted to attain this result, I am certain that many manufacturers, and those not the least important, will be entirely of my opinion.

Under these conditions the question of knowing whether the Le Chatelier test eliminates certain good cements should not even be mentioned by cement makers. The consumers are the best judges of the guarantees with which they should surround themselves, whilst the only part the manufacturers should play is to advise consumers on the possibilities of manufacture.

In the present instance, however, it is possible, without increase of cost, to manufacture a cement which will satisfy the Le Chatelier tests; and it is for the consumers alone, and not the manufacturers—one would think—to take up the matter.

IV.

It would seem, *à priori*, that the fact that cements which, though non-expanding when freshly ground, expand after aeration, is due to the action of the added calcium sulphate on the calcium aluminate present in the cements.

A long time ago Candlot showed that cements which are retarded by the addition of gypsum resume their quickness of setting after being aerated.

According to Camerman, this little recognised phenomenon is the real cause of numerous accidents.

A check specimen of the silicious cement was mixed with 1, 2, 3, and 5 per cent. of its own weight of gypsum. The Le Chatelier test was applied immediately after mixing, and also after storage for fifteen days in the open air.

Eight cylindrical test pieces were prepared from each product, and all the moulds were new and from the same source. After being made, they were kept in water at 17 deg. C. for twenty-four hours, between two sheets of glass, and then immersed in cold water in a water bath, the temperature of which was raised to 95 deg. C. in half an hour, and maintained thereat for four and a half hours.

Results indicate that aeration lessens the expansion of the check specimen and of the cements containing 1 and 2 per cent. of added gypsum, but increases the expansion of the test pieces with 3 and 5 per cent. of gypsum.

The increase, however, is too small for the experiments to be considered decisive, and they consequently need to be completed.

V.

The third objection urged against the Le Chatelier test is lack of precision.

It would perhaps be interesting in the first place to define what is to be understood by precision. There is no doubt that the Le Chatelier test does not attain the precision sought and obtained by Regnault in his experiments; but that it is not at least quite as accurate as the other customary tests applied to hydraulic binding media remains to be proved.

After a series of tests carried out with new moulds from the same source, we have investigated successively:

(*a*) The influence of the width of the shoulders by which the needles are fixed on the test pieces;

(*b*) The influence of the age, or degree of wear, of the moulds;

(*c*) The influence of the method of storage;

(*d*) The influence of the time elapsing between the final setting of the cement and the immersion in hot water.

In the results of a series of tests performed with twenty-five new moulds and four different products, the maximum difference observed in comparison with the mean does not exceed those obtained in the cases of standard tests for hydraulic binding media. Moreover, this difference is unimportant except in the case of low expansion—a circumstance which is not a source of inconvenience, and arises from fixed errors in the reading.

(*a*) As was easily to be foreseen, the moulds with long, thick needle shoulders, more rigid than those with short shoulders, give indications that, for a given cement, correspond to a smaller expansion.

(*b*) We have investigated two methods of storage previous to immersion in hot water.

1. Immersing the test pieces in fresh water for twenty-four hours between two plates of glass.
2. Keeping the test pieces in damp air for twenty-four hours.

In each case, four methods of immersion were practised:*

(*a*) Three hours in boiling water;

(*b*) Five hours in boiling water;

(*c*) Three hours on the water bath at 95 deg. C.;

(*d*) Five hours on the water bath at 95 deg. C.

(*e*) Finally, we have also investigated the influence of the date of the immersion of the test pieces in the hot water.

As a rule the test pieces are immersed twenty-four hours after mixing; but there was reason to think that the results might vary according to the time allowed to elapse between the definitely ascertained final setting and the moment of immersing the test pieces. We carried out four tests with products having different setting times.

VI.

From this series of tests it may be concluded that the form of the needle shoulder, and the age of the moulds, or rather the extent to which they have been used, have an influence on the results obtained, whilst the mode of immersion has but little influence; and that, on the contrary, the results are influenced considerably by the time elapsing between the definitely ascertained completion of setting and the immersion of the test pieces in hot water, at least in the case of products with slow initial setting.

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* In these four modes of immersion the test pieces were placed in cold water, and the temperature raised as quickly as possible (20—30′ at most) to 100 or 95 deg.
The time was measured from the immersion of the cylinders, the length of immersion in the boiling water (or at 95 deg.) being therefore 2½ and 4½ hours.

It is, in fact, characteristic of the Le Chatelier test that it does not give the measure of the absolute expansion at a given moment, but rather the difference between the effort of expansion and the effort of resistance opposed to this expansion by the cement in consequence of its own previous hardening. (See my communication of May 31st, 1902, on comparing the hot water methods of testing.)

May one not, however, base on this fact for the purpose of removing the reproach that the Le Chatelier is excessively severe. It would be sufficient to examine merely whether the delay of twenty-four hours after mixing is adapted for all eventualities, or whether it is not too strict, and that by altering the period to thirty-six or forty-eight hours one would still meet with the inconvenience of eliminating the cements that are stable in cold water. Moreover, in our opinion, the period allowed to elapse previous to the immersion in hot water has nothing to do with the value of the test. It should be left for each user to fix for each class of cement the period that should be allowed to elapse before the Le Chatelier test is applied.

On the other hand, the results obtained having revealed the slight influence of the method of keeping and of immersion of the test pieces in hot water, and the complete concordance of the tests performed with moulds of one and the same origin, it is in our opinion justifiable to regard the Le Chatelier test as being sufficiently accurate in practice and convenient in use. As regards the shape of the needle shoulders, nothing is easier than to control this by regulation, in the same manner that one can fix the number of times the moulds may be used before being discarded.

VII.

In these conditions, one may ask how the German Committee on the setting time arrived at conclusions so opposed to our own.

Truly, when one examines the Tables published by that Committee (Protokoll § 252 and 253) it appears that conclusions diametrically opposed to those actually drawn might be drawn therefrom.

The point that really strikes the eye about these Tables is that all the curves are of the same form, so that laboratory L gives maximum results for all the products, and the laboratory S minimum results.

From this it follows that a systematic error exists between all the laboratories. Since the methods of testing were the same in all cases, and that, as will be seen, the explanation of such discordant results cannot be found in any inexactitude of the method itself, the cause of error must be sought elsewhere.

In our opinion this cause resides in the circumstance that, although the precaution was taken to send the cements to the laboratories in sealed boxes, that of fixing the day for the application of the tests—which ought to have been the same in all cases—was omitted.

Every manufacturer knows, as a matter of fact, that expanding cements lose their expansion when packed in hermetically sealed boxes, and sometimes very quickly, all that is necessary to produce this result being the presence of sufficient water, in the state of moisture or in combination with the acid silicates, to slake the lime still remaining in the caustic state. This water is transferred to the quick lime in consequence of the low dissociation tension of slaked lime at the ordinary temperature.

If the temperature of the cement be raised, this phenomenon sets in very quickly (less than two hours at 100 deg.).*

Hence a difference of several days in the date of the tests may lead to very different results even when the binding medium has been kept in hermetically sealed boxes.

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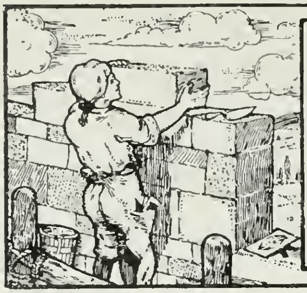
* It is of course necessary that the cements should contain, in one form or another, the water needed to hydrate the lime, in order that they may entirely lose their expansion when kept in hermetically sealed boxes; and cements which do not contain such water remain expansive for an indefinite period. Except by means of delicate tests, it is impossible to know, *à priori*, whether an expansive cement remains so or not.

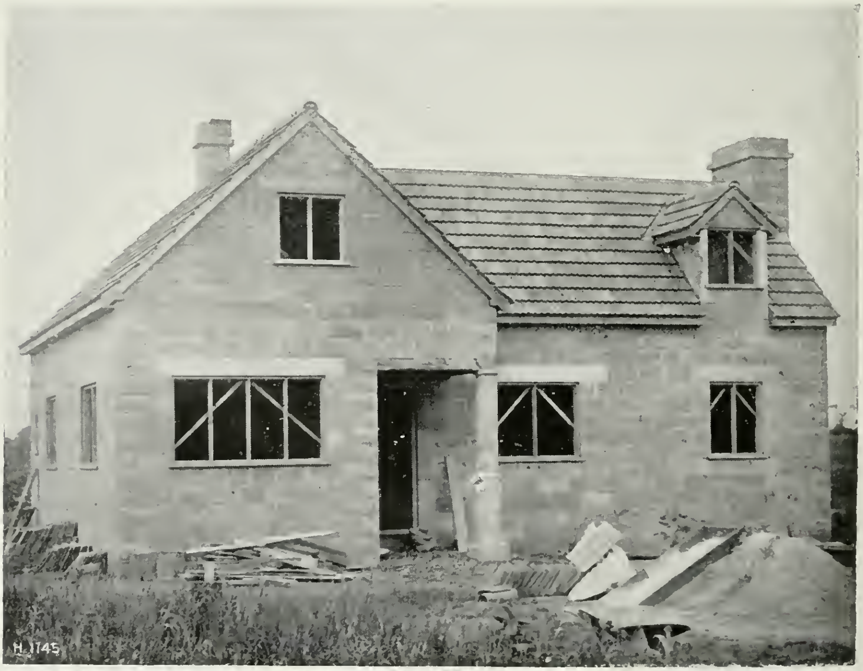


CONCRETE AGRICULTURAL COTTAGES IN NORFOLK.

The cottages described below are built almost entirely of concrete and show in a practical manner the advantages of this material for cheap cottages, and we would commend the perusal of this article to those interested in housing problems.—Ed.

In connection with the rural housing question which has been so much before the public of late, wide-spread interest has been created in Norfolk by the erecting of cheap concrete cottages by the Wayford Tenants, Ltd., on their estate at Wayford in the parish of Stalham, N.E. Norfolk. In comparison with the existing cottages in the district, which are of the usual type of poorly-lighted, badly ventilated, unsanitary farm labourers' cottages, the new concrete dwellings show to a remarkable advantage from the point of view of efficiency and economy.

The cottages shown are built practically entirely of concrete. The walls are of concrete blocks 18 in. by 9 in. by 4½ in. All external walls are



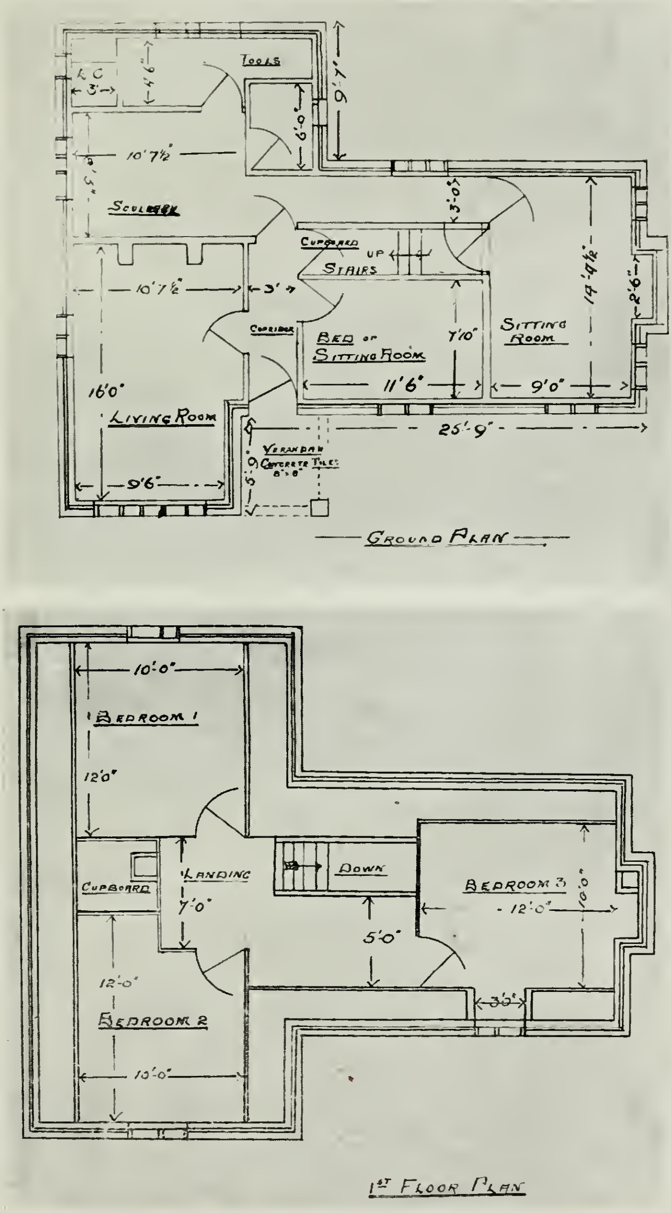
Cottage, Type 2, in course of construction.
Concrete Agricultural Cottages in Norfolk.

10 in. in thickness. This allows for a one-inch cavity between the two $4\frac{1}{2}$-in. blocks. It adds to the comfort of the house, ensuring the inside walls being absolutely dry and a more even temperature.

The partition walls are all built of blocks $4\frac{1}{2}$ in. thick. The floors are kept well above the ground level. The height between floor and ceiling is 7 ft. 10 in. throughout, and there is at least one large casement window in each room, so arranged that it is possible to have a part open without inconvenience during the severest weather.

A special feature of the cottages is the roofing tiles, which are absolutely weather-proof. They are made of concrete, as are also the flooring tiles or quarries. Both of these being glazed, give the buildings a neat finish. These tiles work out considerably cheaper than the ordinary flat or pin (clay) tile.

The cottage shown in our illustration is of bungalow type, the rooms are commodious, and the roof is so arranged that two bedrooms could be made in it with little expense if



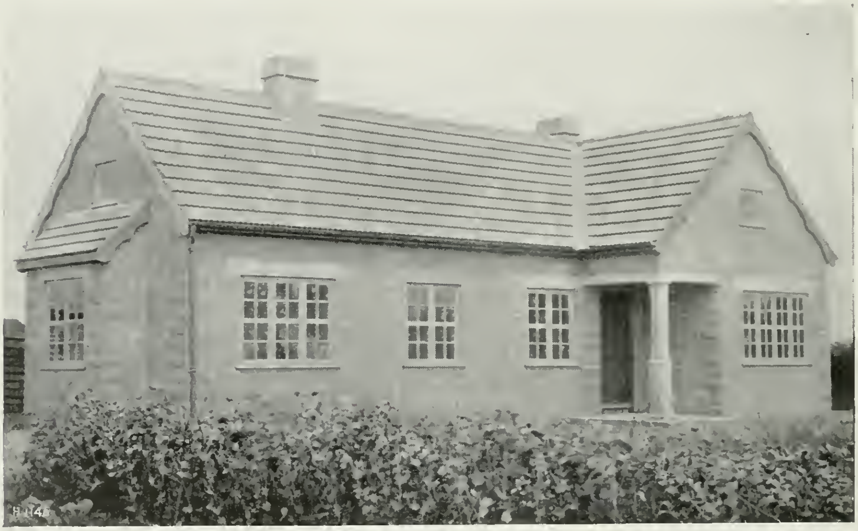
Ground and First Floor Plans.
CONCRETE AGRICULTURAL COTTAGES IN NORFOLK.

required; but under the present arrangements it will be utilised for a storage loft.

The size of the rooms are as follows:—Living-room, 16 ft. by 11 ft.; kitchen, 11 ft. by 9 ft.; larder, 5 ft. by 4 ft.; three bedrooms, 14 ft. 6 in. by 10 ft., 11 ft. by 9 ft. and 11 ft. by 8 ft. respectively, with a coal store 6 ft. by 5 ft.

Each room is well lighted and ventilated. The ceilings are of "Beaver Board," which is composed of pure wood pulp, and which gives the appearance of a panel ceiling when finished.

At the back of each house there is an underground tank for the storage of rain-water. This is built of concrete blocks made specially for this purpose. The cover is of reinforced concrete, with a manhole in the centre. A small pump is connected from this for the use of rain-water in the house. The size of the tank is 6 ft. diameter by 6 ft. deep. Thickness of blocks 4 in.



View of Cottage, Type 1.
Concrete Agricultural Cottages in Norfolk.

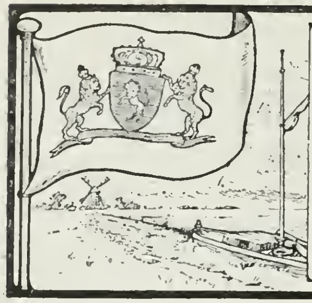
Special consideration has been given to an efficient drinking-water supply for the cottages, of which there will eventually be fourteen or more. The supply main is 2 in. diameter and is fed from a concrete reservoir situated on the highest point of the property. This is fed from a well some two hundred yards distant from the reservoir. This well is 6 ft. in diameter and is also formed of concrete blocks. The reservoir is 20 ft. square by 5 ft. deep, the walls being 9 in. in thickness. Forty tons of concrete were used in its construction, and the work was completed in one day. The capacity of the reservoir is 10,000 gallons, and the natural elevation gives sufficient fall to obtain the required pressure to supply the fourteen cottages to be erected on the estate.

The cost of the cottage illustrated here is £185 complete. By the use of blocks a saving of 75 per cent. in the jointing material is effected. Less plaster is also required for the walls, as the surface is less rough than in the case of bricks.

It must be mentioned that all blocks, roofing tiles, and quarries are made on the estate. These particulars of what is being accomplished on the Wayford Estate should convince anyone that cheap and desirable cottages can be erected of concrete on this system much more economically than by the use of bricks. This should be a matter for the consideration of the Departmental Commission of the Rural Housing which has been lately appointed by the Board of Agriculture.

The machines on which the concrete floor tiles, ridge tiles, and inter-locking roofing tiles were made were supplied by Mr. R. H. Baumgarten, of 8, Manor Park, Lewisham, London, S.E.

The whole of the work is under the personal supervision of Mr. B. F. Hartley, the works manager, who is responsible for the design of the reservoir and the moulds for the concrete blocks, etc.



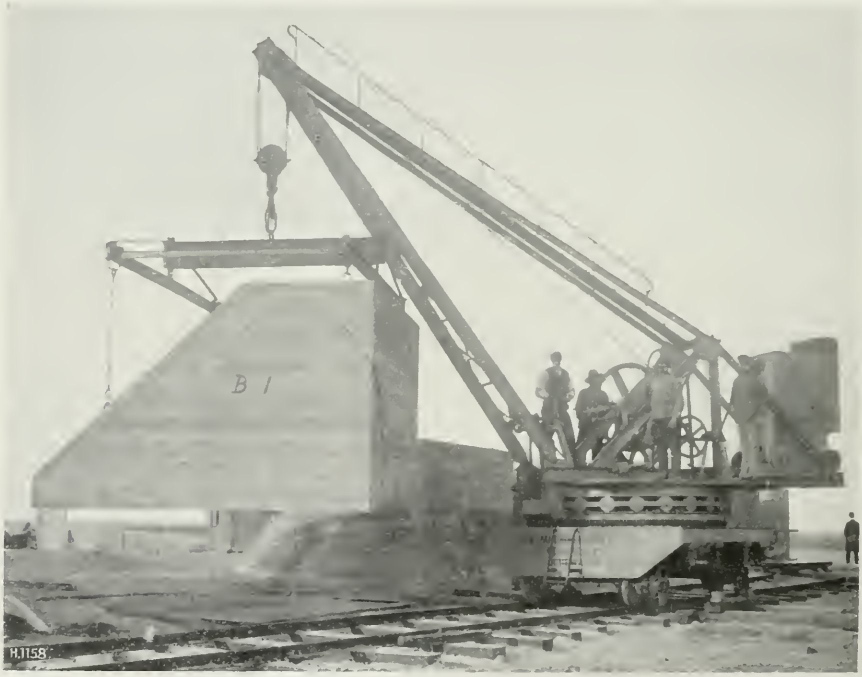
APPLICATIONS OF REINFORCED CONCRETE IN THE CONSTRUCTION OF VARIOUS MARITIME WORKS IN HOLLAND.

By V. DE BLOCQ VAN KUFFELER,
Engineer of the Waterstaat, Hoorn, Holland.

The following paper on Reinforced Concrete Sea Work in Holland was presented to the XIIth International Congress of Navigation recently held in Philadelphia.—Ed.

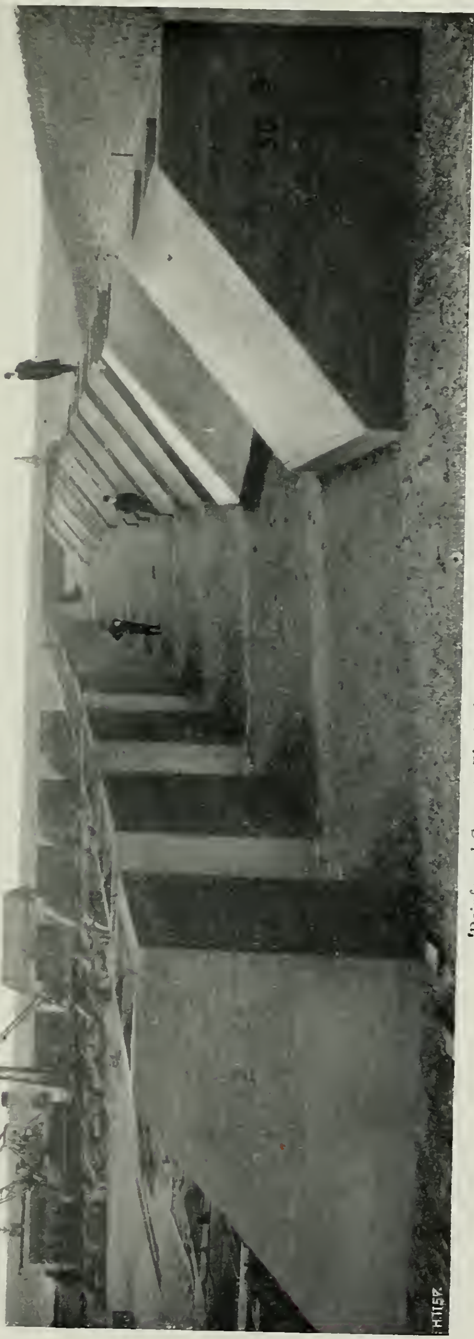
This paper is confined to work carried out on breakwaters or reinforced concrete works exposed to the action of the sea, and no mention is made of works executed in maritime ports which are situated some distance from the sea coast, such as at Amsterdam and Rotterdam.

Amongst recent works on breakwaters in our country the most interesting are the extension of the south jetty of the Hook of Holland (outlet which connects Rotterdam with the sea) and the improvement of breakwaters in the fishing port of Scheveningen.



Conveying Reinforced Concrete Blocks to the Breakwater, Scheveningen.
Reinforced Concrete Sea Works in Holland.

Another small reinforced concrete lighthouse has also recently been built on one of the jetties at the mouth of the " Ysel " river on the Zuyder Sea.



'Reinforced Concrete Blocks for the Breakwater, Scheveningen.'

REINFORCED CONCRETE SEA WORKS IN HOLLAND.

As regards the construction of landing stages and wharves exposed to the sea, reinforced concrete has been employed in a small landing stage in the fishing port of Ymuiden, and for the foundations of a quay wall at Vlieland. We also find reinforced concrete applied on a fairly large scale for the protection of banks and foreshores, such as the groynes built by Engineer de Muralt for protecting the northern coast of Holland, and the island Vlieland and a groyne on the island of Urk in the Zuyder Sea.

WORK ON BREAKWATERS.

Mr. van Kuffeler gave a description of the method of construction adopted in the extension of the south jetty at the Hook of Holland. The method employed had been used with great success for improving the inland channel of the port of Enkhuizen on the Zuyder Sea. It consisted of placing a layer of fascine mattresses across the whole width of the jetty and as much sand as possible run in between them. Above ordinary low-level water the jetty consists of fascine mattresses covered with a revetment of basalt blocks of about 0·55 m. by 0·30 m. This revetment will bear against oak piles 5 m. long, and a double row of oak piles 3·5 m. long will be driven through the revetment. The work which has been carried out at the Hook shows the important *rôle* which is played by sand when it is deposited under water and the economy effected by this method of construction.

Improvement of the Breakwaters of the Fishing Port of Scheveningen.—To provide against the displacement which has taken place in the blocks which form the breakwater at Scheveningen, and which are deposited up to ordinary high-water mark, it has been necessary to employ very heavy blocks, similar to those used in the port of Ymuiden. The breakwaters had to be levelled flush with low-water mark in order to enable these blocks to be deposited, and as the breakwaters are built of basalt rubble they provided a sufficiently sound foundation for these large blocks. The latter are hollow and are made in reinforced concrete 4 metres long, 3·6 metres wide, 2·6 metres deep, weighing 10,000 kilos. They are filled with 22 cu. metres of concrete and when full weigh about 56,000 kilogrammes. The hollow blocks are placed by means of a 10-ton crane. Shortly after they were deposited a few of these blocks were shifted about 1 metre out to sea, but since then no displacement has taken place.

Some concrete blocks 4 sq. metres in area, weighing 700,000 kilogrammes, were then sunk in front of the large blocks for a length of 110 metres along the jetty.

The sides of the hollow blocks are 8 cm. thick, and they are strengthened in the middle by a rib 25 cm. wide. The main reinforcement consists of 8 mm. rods, and the secondary reinforcement of 6 mm. rods, all spaced 10 cm. apart. Some 16 mm. rods are used for the reinforcement at the corners.

The concrete used in the hollow blocks consists of 1 part of cement, 1½ parts of sand and 1½ parts of gravel, and the mass concrete for filling consists of 1 part of cement, ½ a part of trass, 3 parts of sand, and 5 parts of gravel.

The reinforced concrete of the breakwaters has given complete satisfaction. The work was carried out by the Hollandsche Maatschappy tot het maken van Werken in Gewapend Beton, of the Hague.

Reinforced Concrete Lighthouse on the South Jetty at the Mouth of the Ysel on the Zuyder Sea.—The lighthouse is built upon a foundation which consists of a block of concrete of about 24 cu. metres, which bears upon sixteen piles driven through the jetty.

This concrete foundation block is reinforced at the top to avoid damage by frost during winter, and is reinforced in the middle for anchoring the columns which carry the lighthouse. The concrete used in this is made up of 1 of cement, 1⅔ of sand and 3⅓ of gravel.

The lighthouse itself is built of reinforced concrete and consists of a square cabin 1·7-metre side and 2·4 metres high, which is carried on four columns with rounded arrises. There are two platforms carried on brackets; one in front of the door is connected to the jetty by an iron staircase, and the other on a level with the roof, facing seawards, which carries the flashlight.

The sides of the cabin are at least 10 cu. metres thick and the columns are 20 by 20 cu. metres. Concrete used for this part of the work is made up of 1 of cement, 1⅔ of sand, and 2⅖ of gravel.

No damage has been caused so far by ice or by storms. This method of construction is very simple and is strongly to be recommended in the case of any structure which is not exposed to very heavy waves.

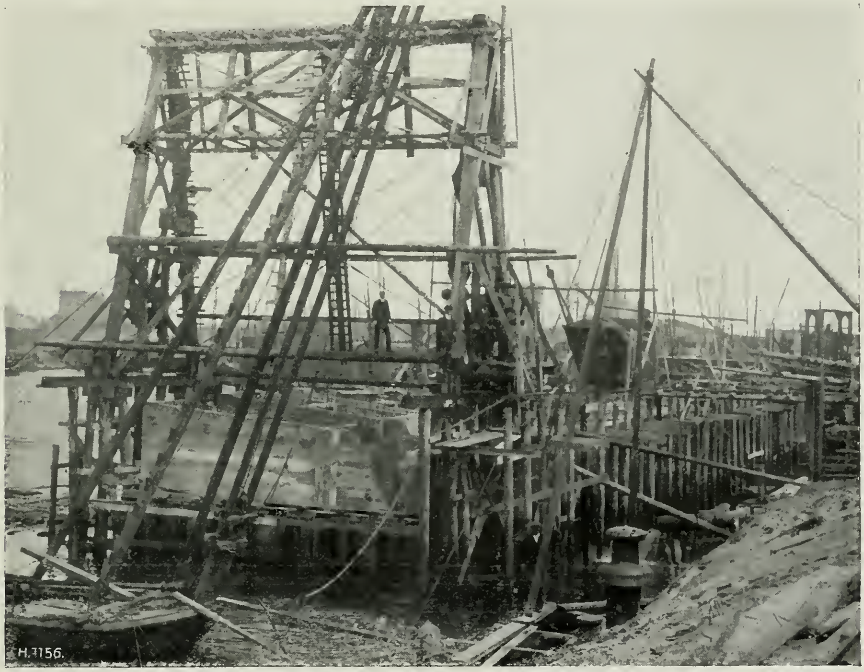
WHARFING.

Open Wharf at the Fishing Port of Ymuiden.—The open wharf at the fishing port of Ymuiden was built in 1910, on a system of buttressed walls which were constructed in a series of pits which are 3 metres apart and are connected to one another at the back and the top by two sets of slabs.

The earthwork is at the level of + 3 metres, and the bottom of the port is at a level of −6 metres.

The pits are rectangular in section, being 3 metres wide along the line of wharfing and 10 metres long at the bottom and 6 metres long at the top. The openings in the pits are closed at the top by a horizontal slab and in front by a vertical slab, these being connected together. They are strengthened by buttresses which rest upon beams built over the pits.

A platform runs right across the pits and the intervening spaces, and is connected to a vertical retaining wall strengthened by buttresses.



Reinforced Concrete Quay Wall at Ymuiden, in course of construction.
Reinforced Concrete Sea Works in Holland.

The whole of the wharfing is built of reinforced concrete, but the coping is of granite. The pits are filled with mass concrete, made up of 1 of cement, 1 of lime, 3 of sand, and 5 of gravel.

After they have set sufficiently the pits, which have been built on temporary scaffolding, are lowered very gradually by a winch right to the sea-bed, which has been dredged beforehand to the level of −6 metres. It was necessary to build slabs to close the spaces between the pits after they were sunk, having due regard to the slope or batter of the sides of the pits. The concrete used in the pits and slabs is made up of 450 kilogrammes of cement per cu. metre of concrete, and contains equal quantities of sand and gravel. The framework of the pits is reinforced with 12-mm. and 25-mm. rods, the main reinforcement consisting of 10-mm. and 12-mm. rods, spaced 10 cms. apart, and the secondary reinforcement of 8-mm. bars, spaced on an average 20 cms. apart. There is slightly more reinforcement on the slabs than in the piers. There are no particular features about the superstructure, which was built after the slabs had been placed, and connected to the latter and to the pits. The concrete used contains 400 kilogrammes of cement per cu. metre of concrete, and is made up of 3 of sand and 5 of gravel. This work was also carried out by the Hollandsche Maatschappy tot het mahen van Werken in Gewapend Beton, of the Hague.

Quay Wall at Vlieland.—Vlieland, one of the islands in the north of Holland, in the North Sea, contains a small port surrounded by weak quay walls. Considerable masses of blocks of basalt, required for the reconstruction of the foreshore protection, are found lying on the embankments, and their weight caused the quay wall to shift to such an extent in 1909 that its reconstruction became imperative.

It is the general custom in Holland to construct the walls upon piles in cases of this kind, but this method of construction would have been quite expensive at Vlieland because the foundations required protection against the teredo.

The wall would in this case have had to be in a perfectly dry trench—a very difficult operation, on account of the percolation of water from the neighbouring dunes.

The adoption of reinforced concrete has enabled us to avoid all these difficulties, because a foundation in reinforced concrete can remain above ordinary low water, whereas a foundation in wood must remain under water.

The top of the foundation is 0·60 metre above the level of low tides, so that it could be built without a coffer dam. This method of construction showed a saving of 32 per cent.

The quay wall proper is made with basalt from the old wall.

The quay wall rests upon a reinforced concrete raft 3·5 metres wide, which is supported by a row of sheet piling, and by three rows of timber piles, 1·2 metres apart. The timber piles are connected at the head by reinforced concrete beams, which are connected to the beam along the top of the sheet piling, this arrangement and the raft holding the wall together in the longitudinal direction. Thanks to the neighbouring dunes, the timber piles behind the sheet piling are always quite moist, and therefore no rotting is to be feared. The sheet piling is 5·40 metres long, and is moulded horizontally; it was driven with an ordinary pile driver and a jet of water under pressure.

The top of the sheet piling was broken after driving, so as to connect its reinforcement with that of the cross-beams and raft, both of which were moulded subsequently.

The sheet piling 0·20 metre by 0·12 metre is reinforced with six 9-mm. rods, and the longitudinal beam by seven 9-mm. rods, the cross-beams by four 17-mm. rods, and the raft by 9-mm. rods, spaced 0·08 metre apart as regards the main reinforcement, and spaced 0·18 metre apart as regards the secondary reinforcement.

The concrete consists of 1 of cement, ½ of trass, 3 of sand, and 4½ of gravel.

The raft and beams, although protected as far as possible by the shuttering, were exposed to the action of sea water from the very outset, but notwithstanding this, no serious damage arose through the force of the waves, and the marks of the shuttering are still visible at the present moment. Although the concrete is in good condition in every part of the structure, one can now see that the portion which was made *in situ* and exposed to the chemical action of the sea water is not so hard as the portion of which the piles were made, as these were allowed to set in a moist atmosphere before being used. In future a stronger mix will be specified for the concrete of the beams and raft (1 of cement, ½ of trass, 2½ of sand, and 3½ of gravel).

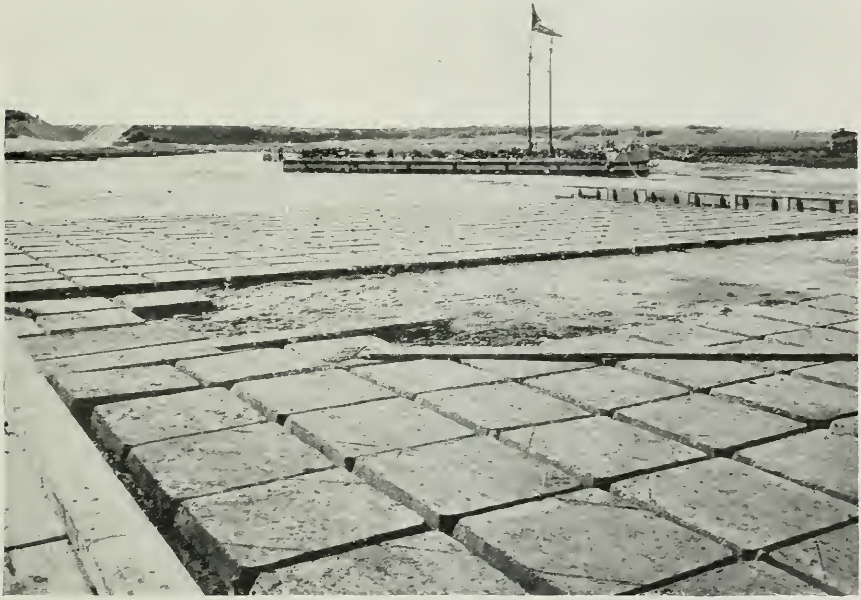
PROTECTION WORK OF JETTIES, EMBANKMENTS AND FORESHORES.

Works carried out by Engineer de Muralt.—At the last Congress a paper was read which described some applications of reinforced concrete by Mr. de Muralt, which were as follows :—

1. The revetment of banks by the slab and beam system.
2. The raising of embankments.
3. The reconstruction of the Ossenhoof spur or groyne.
4. The substitution of reinforced concrete for fascine mattresses.

Since the publication of Mr. Cool's report Mr. de Muralt has greatly improved this last method of applying reinforced concrete, and has invented a very ingenious revetment for banks and foreshores which are not exposed to very violent storms.

In this system of protection the concrete blocks or tiles are 0·95 metre long, and are reinforced at the sides with $\frac{5}{16}$-in. rods, the ends of which protrude at each corner of the slab and are bent round in the form of a hook. The sides of the blocks are also reinforced diagonally with two $\frac{3}{16}$-in. rods, which form a cross. The blocks vary in section according to the kind of bank they have to protect.



De Muralt Coast Defence Work.
Reinforced Concrete Sea Works in Holland.

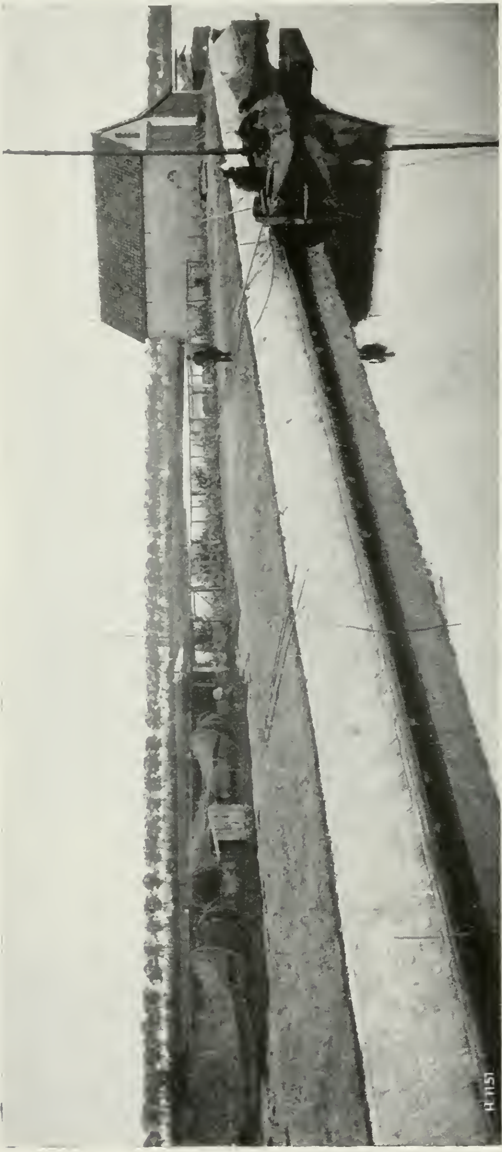
The blocks are laid on the beach during low water at a distance of 1 metre from centre to centre and are connected together by the attachments at the corners.

Trestles are erected on the beach above the spot where the blocks are being assembled, and a specially constructed pontoon is floated out to the trestles during high water. These support the platform fore and aft, so that the centre deck of the pontoon is vertically over the mattress of blocks. A ring bolt is inserted at every second attachment of the blocks, and a wire rope is attached to these ring bolts at one end and at the other end to winches on the pontoon.

As soon as the pontoon is floating on the rising tide, it is towed by a tug with the mattress of blocks to the site, where it is anchored, and the mattress is lowered horizontally, until it touches bottom.

The concrete is made up as follows: 1 of cement, $\frac{1}{6}$ of trass, $2\frac{1}{2}$ of sand, $3\frac{1}{2}$ of gravel, for heavy blocks; and 1 of cement, 1-9th of trass, 1 2-5th of sand, and $2\frac{1}{3}$ of gravel for the light blocks.

The maximum size of any mattress which has hitherto been deposited by means of two pontoons linked up is 400 sq. metres. These dimensions are small compared with those of ordinary fascine mattresses, which can be made up to 2,500 sq. metres in area or even more. One can, of course, mould reinforced concrete platforms on a much larger scale if larger pontoons are used, but the construction then becomes more complicated. Reinforced concrete mattresses can, however, be deposited at a much greater rate than fascine mattresses, because their higher specific weight allows them to be lowered irrespectively of the speed of the current, and even during quite heavy weather.



De Muralt Coast Protection Work.
REINFORCED CONCRETE SEA WORKS IN HOLLAND

There is always a gap between the blocks in a reinforced concrete mattress, but this is not objectionable, as a rule, except where the currents are very swift and the gaps have not been filled in.

Revetments of reinforced concrete mattresses require a fairly regular slope of bank, because where the ground is broken or uneven the blocks do not settle evenly, and they break.

The steel reinforcement which connects the blocks together may be quite strong enough at the outset, but in time it is liable to become affected by the sea water, and in that case any pits or holes which occur under the blocks may have disastrous consequences.

As the thickness of reinforced concrete mattresses is comparatively small, they cannot materially assist in raising the height of the bank.

Although reinforced concrete mattresses cannot evidently be substituted in every instance for fascine mattresses, they offer very special advantages, especially as regards the precision with which they can be deposited independently of the direction or strength of the current, and of the weather, and also the advantage of a relatively low cost.

This new method of construction, which is both practicable and ingenious, deserves very serious consideration for the protection of under-water slopes.

Mr. de Muralt has devised a very simple but efficient method for the protection of banks which are not very exposed. The whole of the surface of the bank is faced with concrete blocks 0·40 metre square. These are laid with each alternate block overlapping the lower surface of the adjacent block by 4 cm.; the total thickness of the blocks varying from 6 to 8 cm.

The alternate blocks which overlap the others and key the system to the bank are attached to the latter by a reinforced concrete key 40 cm. long. This key is tapered and passes through a tapered hole in the slab; its dimensions are 0·13 by 0·13 metre at the top and 0·08 by 0·08 metre in the middle.

When the blocks are in place, holes are made in the earthwork by means of a boring tool, and the reinforced concrete key is subsequently driven through the hole, to the necessary depth. This method avoids any risk of breaking the head of the concrete key, and enables the revetments to be securely fastened to the bank. The concrete employed is generally made up of 1 of cement, ½ of trass, 2½ sand, and 4 of broken basalt or Quenast porphyry.

The key and the alternate blocks which have a tapering hole are reinforced with 6-mm. rods. The outer edges of the top and bottom rows of blocks are protected and held fast by longitudinal beams driven about 0·40 metre into the soil.

From a technical point of view the new system affords the great advantage of a revetment made up of small units, which enables it to follow the contour of the ground without risk of pits or potholes forming underneath. If the bank should shift too much the revetment can be taken up in parts, and re-laid at a very slight expense.

The advantages of this new system of facing are evident, but it is only in time that we shall know what effect the action of the waves has upon banks so protected. Up to the present no damage has occurred, and as the revetment hardly offers any resistance to wave action I do not think we need have much fear of any damage arising.

Groynes for Protecting the Northern Coast of Holland.—In general, groynes are built in our country by means of fascine mattresses covered with basalt rubble, and very often these are supported on rows of piles. This method is fairly costly in construction and up-keep; especially for the portions above the level of ordinary high tides. Consequently those who have had to build new groynes have sought a less costly substitute for this method, and this has been found in the form of reinforced concrete. The first groynes of reinforced concrete were built in 1910 on the northern coast of Holland at Groote Keeten, and on the island of Vlieland.

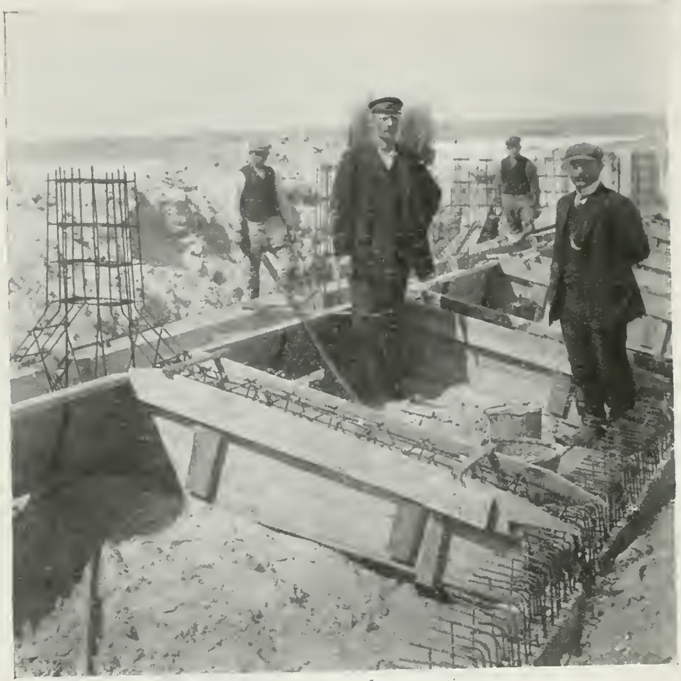
Until then, some parts of the existing rubble revetment had been replaced by a reinforced concrete facing, but nowhere had the whole structure been re-built in reinforced concrete.

Groynes should be able to withstand the most violent hurricanes. To achieve this result:

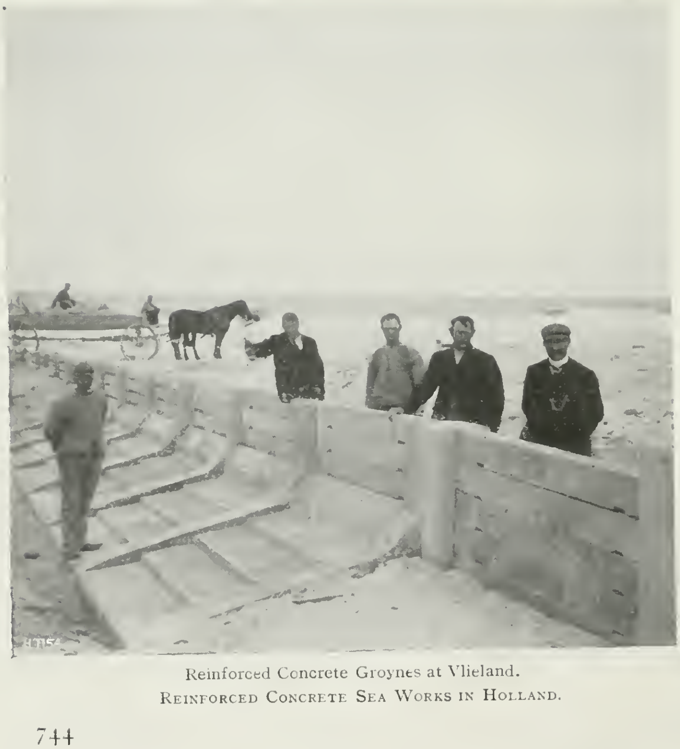
1st, The foundation of the groynes should be sufficiently deep in the foreshore.

2nd, The revetment should be sufficiently strong.

The foundations of the groynes at Groote Keeten consist of two parallel rows of beams of 1 metre deep, and 0·30 metre wide, embedded in the shore, and provided at the top with a groove 0·15 by 0·15 metre to hold the facing slabs, which are 18 to 28 cm. thick, 3·5 cm. long and 4·7 cm. wide, in position. The beams are 4 metres long. The two rows of beams are connected every 4 metres by a cross-beam, which has rebates to hold the slabs in position. The reinforced concrete portion of each groyne above the level of ordinary high tides is 68 metres long.



Groynes in course of construction.



Reinforced Concrete Groynes at Vlieland.
REINFORCED CONCRETE SEA WORKS IN HOLLAND.

The longitudinal beams are reinforced with seven 1·6-mm. rods, and the slabs are reinforced with 10-mm. rods spaced 10 cm. apart for the primary reinforcement and 20 cm. apart for the secondary reinforcement.

The cross beams are reinforced with five 20-mm. rods and also with a secondary reinforcement of 16-mm. rods spaced 10 cm. apart. The concrete is made up of 1 of cement, $\frac{1}{3}$rd of trass, $\frac{1}{3}$rd of sand, and 2 $\frac{2}{3}$rds of gravel.

This method of foundation suffices wherever there are no great differences in level in the shore.

The easier and simpler the method of carrying out the work, the greater is the scope of application of reinforced concrete, and it is particularly on the lower part of the shore that the economy realised in reinforced concrete construction is greatest.

Groynes on the Island of Vlieland.—The construction of groynes at Vlieland differs considerably from that of Groote Keeten, as both foundations and revetment are carried out in a different manner.

The foundations consist of two parallel rows of sheet piling connected at the top by a longitudinal beam, which rests on cross-beams spaced 2·70 metres centre to centre. The space between the beams is covered with slabs which rest on the ground. The use of a single slab across the groynes has been abandoned, as it would have had to be saddle-shaped and might have been broken if it did not bear quite flat upon an uneven soil. The slab is therefore now made in two parts, which are held down by a beam with rabbets or flanges.

All the beams are flanged or rabbetted so that they act also as a framework for the slabs which they hold in position. The upper surfaces of the slabs should be stepped, so that, as the water falls off the groyne, it shall not wear away the beach.

The slope of the Vlieland foreshore amounts to as much as 1·5, and it would be advisable if the groyne could follow the rise and fall of the beach. Since this is manifestly impossible, one can provide the groyne with a movable superstructure, although an arrangement of this kind is only feasible on the portion of the foreshore near the dunes, where the wave action is never very violent.

At Vlieland, columns 0·60 metre high were erected on the cross-beams. These columns have grooves into which three planks 0·20 metre high can be inserted.

The structure is of reinforced concrete throughout. Concrete consists of 1 of cement, ½ of trass, 3 of sand, and 4½ of gravel.

The sheet piling is 2 metres long, 12 cm. thick, and 20 cm. wide, with a reinforcement of six 6-mm. rods which protrude from the concrete. They are driven very easily by water under pressure.

The reinforcement of the longitudinal beams consists of seven 6-mm. rods which are connected to the reinforcing rods of the sheet piling, and also to the primary reinforcement of the cross-beams, which consists of seven 8-mm. rods.

The longitudinal beams are 0·20 metre by 0·22 metre, and the cross-beams are 0·26 by 0·36 to 0·50 metre, and the columns 25 by 36 cm. The columns are reinforced with twelve 8-mm. rods, and the reinforcement of the slabs, which are 0·10 metre thick on the average, consists of No. 15 gauge Expanded metal.

A groyne on the Groote Keeten system is now being built on the island of Texel, but it has sheet piling instead of 1-metre beams.

In general the system adopted at Vlieland is to be preferred whenever the execution of such fairly complicated structures does not present any special difficulties, such as in the neighbourhood of dunes. When one is satisfied that the amount of reinforcement of the revetment slabs on the Groote Keeten system is sufficient, this system will be preferred on account of its simplicity, for it will enable groynes to be built up of separate units which can be made elsewhere than *in situ*. This will be most important, not only for the concrete, which will no longer be exposed, whilst it is being made, to the percolation of sea-water, but also principally for the increased scope which this form of construction will obtain, since no further objection can then be raised against making the part of the groynes which lies below the level of ordinary high tides, of reinforced concrete.

REMARKS CONCERNING THE EFFECT OF SEA WATER ON REINFORCED CONCRETE.

The large number of applications of reinforced concrete in the structures described above and their fairly frequent application in structures which have been described for previous Congresses testify to the increasing confidence which this new method of construction is gaining, even for works exposed to sea-water. It is, however, essential to acquaint oneself pretty thoroughly with the effect which the sea-water has upon this material.

It is admitted by everyone that the steel embedded in the concrete must be completely encased, and the steel must not be too near the surface. A minimum covering of 2 cm. should be specified, especially for maritime work.

In these circumstances it merely resolves itself into a question of the extent to which the concrete itself is affected by sea-water.

The decomposition of the concrete is generally caused by the salts which are in the sea-water, and the action of the sulphates is especially to be feared when these come in contact with unstable combinations such as the lime formed during the long period the cement takes in setting. Clearly, it is during the first period of the work that the action of sea-water is most harmful, for it is during that time that the combinations referred to are the least stable. This theory is certainly borne out by practice.

Some experiments carried out by a special committee at Sylt in Germany, and also at the testing station of Gross Lichterfeld, show that the resistance of concrete which has been allowed to set in a moist atmosphere for one year before being placed in sea-water, is greater than that of concrete which has been exposed to the action of sea-water only a short time after its manufacture, and this applies particularly to any mixtures which are rich in cement.

As I have remarked, the concrete in the sheet piling of the quay wall of Vlieland, which was allowed to set in a moist atmosphere before being put in place, is harder at present than that of the horizontal beam of the same mix, but which was exposed immediately it was made to the percolation of the sea-water.

The mix of the concrete used in the jetties of the fishing port of Scheveningen was the same in the monolithic portion of the structure as in the foundation blocks which were cast on the dunes; nevertheless the blocks are in perfect condition, whereas the concrete used in the monolithic portion of the structure already shows some traces of disintegration.

In order to ensure the preservation of reinforced concrete in sea-water it is recommended to allow it to set as far as possible in a moist atmosphere before being used on the site.

Where concrete is exposed to the action of sea-water as soon as it is made, the mix must be richer, so as to increase the resistance.

The amount of water used for the manufacture of concrete is comparatively small, so that the quantity of salts introduced into the concrete when sea-water is used is negligible, compared with the quantity of salts introduced into the concrete by diffusion when the latter is placed in sea-water, as this water holds a large amount of salts in solution.

The use of sea-water in the manufacture of concrete has therefore very little deleterious effect.

The more difficulty there is in the diffusion of the satured water, the less is the concrete affected, and therefore the latter must be water-tight in maritime works, but at the same time it is advisable that the cement should be in such condition as not to be affected too much by the sea-water.

The question of the quality of the cement in maritime works is of the greatest importance, especially if the rate of setting has to be very regular. One should only employ slow-setting Portland cement whose ingredients have been thoroughly mixed before being burnt in the kiln.

It is the more unstable combinations of cement which are most easily attacked by sea-water, and when materials like trass or cement pouzzolanes are added to it, a portion of the lime will combine and form a silicate, whilst the other combinations will become more stable and more resisting to the action of the sea-water. As the process takes place very slowly the effect of the trass will be greater if the concrete is allowed to set beforehand in a moist atmosphere.

When fine trass is used the mortar becomes still more water-tight, and the addition of trass to the cement increases the strength.

Thanks to the assistance of my colleagues, I have been able to collect the data regarding the compositions of the concretes, cements and trass which have been used in our country for maritime work, and we gather from this that concretes whose mix is weaker than 1 part cement, ½ part trass, 3 parts sand will not give satisfactory results in sea-water. Concretes of this mix, however, are found to be in perfect preservation when they have been allowed to set in a moist atmosphere before being placed in position. (For instance, the blocks in the jetties at Scheveningen have been exposed to the action of sea-water during the last ten years, and show absolutely no traces of disintegration.) This result is remarkably in accordance with the results of the German tests.

The mix of 1 part cement, ½ part trass, 3 parts sand may still lead to disappointment when it is exposed to the percolation of sea-water whilst it is being made (the monolithic portion of the jetties of the port of Scheveningen), and consequently a richer mix is advisable, such as at least 1 part cement, ½ part trass, 2½ parts sand.

When no trass is used a richer mix should be specified. The German tensile tests show that the strength of the mortar of 1 part cement, 2 parts sand goes on decreasing from the first to the fifth year, and consequently one is led to suppose that this mortar will not be suitable for sea-water, and that a mix of 1 part cement and 1½ parts sand is necessary.

The concrete used in the construction of the jetties and breakwaters of the port of Ymuiden was made of 1 part cement, 3 parts sand, and 5 parts gravel, but it has not given quite satisfactory results. In this case, however, they had to contend with the hammering action of the blocks of the breakwaters which had been shifted during storms, and which were an additional destructive force to the action of the sea water. The concrete used in the blocks employed for the maintenance of the works up to 1905 consisted of 1 part cement, 1½ parts sand, 3 parts gravel, with most satisfactory results. Subsequently a cement-trass concrete, composed of ⅓ part trass, 2 parts sand, 3⅔ parts gravel was used. As to the quantity of trass that one should employ, I think one should recommend ½ part of trass for 1 part of cement.

The kind of sand used has no doubt some effect upon the quality of the concrete, but the fine sand from the dunes may give perfectly satisfactory results. Its mixture with the cement-trass is, however, slightly more difficult, and requires more working.

The voids in the sand must be filled by the cement-trass, so that the mortar shall be water-tight; consequently, sands which leave few voids are generally to be preferred.

As the concrete must be water-tight for maritime works, the quantity of mortar used must be sufficient to fill the voids in the gravel.

If the reinforcement is very complicated it is advisable to diminish the quantity of gravel slightly, so as to be quite sure that the reinforcement and the aggregate are well embedded in the mortar.

Aggregates which cannot pass through a 0·20 cm. mesh should be rejected for reinforced concrete, unless the reinforcement is placed at a fairly good distance from the surface.

Wherever the strength of the materials of construction is less, the homogeneity of their combination must be increased. This applies equally to reinforced concrete, and especially to reinforced concrete exposed to the percolation of sea water.

The materials for the concrete must be thoroughly mixed with the greatest possible care, and the concreting must also be carried out with great care. The shuttering must be sufficiently strong to resist being put out of shape by the concrete during concreting; its dimensions must be carefully seen to, and it must be water-tight.

Any portion of the concrete not properly mixed is liable to be affected by the sea water, which can then penetrate to the reinforcement and attack it.

The more the concrete is well rammed the less porous it is, so it is advisable to avoid putting too much water into the concrete during the mixing. It should, however, be sufficiently plastic to allow the reinforcement to be well embedded in it. The quantity of water to be used depends on the kind of structure that is being erected.

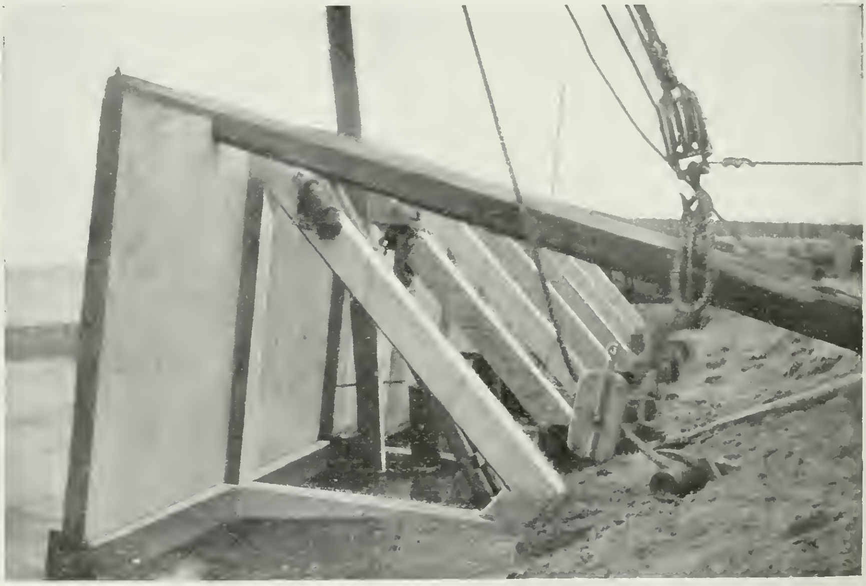
If, on removing the shuttering, the concrete shows a rough surface, it is advisable to face it over with a thin coat of mortar of the same proportions as in the concrete. In that case it is necessary that the concrete should be damp and the mortar so fluid that it can be used with a brush.

Care should be taken that the concrete is kept damp whilst it is setting in the air, as the cement requires moisture to assist its chemical action.

Briefly, I am of opinion that reinforced concrete in our country is sufficiently safe to be applied to maritime work, provided one keeps strictly to the following rules:—

1. Only to employ slow-setting Portland cement of the best quality, which sets very regularly.
2. To thoroughly mix the concrete whilst it is being made, and to employ the greatest possible care in moulding the concrete in strong water-tight shuttering.
3. Preferably to use trass (or pouzzolanes).
4. To use a water-tight concrete, and a mix which shall not be less than the following proportions: 1 part cement, 1½ parts trass, 3 parts sand, or 1 part cement, 1½ parts sand.
5. As concrete allowed to set in a moist atmosphere before it is used *in situ* has greater resistance to the action of sea water it is always advisable where possible to use a richer mixture if the concrete is to be exposed to the percolation of sea-water whilst it is being made and before it has been allowed to set.

In concluding my report, I should like to remark that, in my opinion, a suitable mixture, very careful manipulation, the use of a good brand of cement and of trass (pouzzolanes), and setting in a moist atmosphere, are the most efficient means of ensuring the preservation of the reinforced concrete in our maritime works.



Construction of a Reinforced Concrete Groyne on the Island of Urk.

Reinforced Concrete Sea Works in Holland.

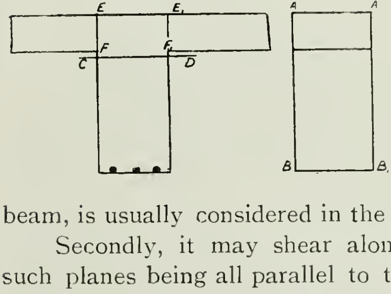
SHEAR FORCE IN T-BEAMS.

By JOHN A. DAVENPORT, M.Sc., A.M.Inst.C.E.

In our issue of July last we published an article by Mr. Davenport on "T-Beam Design," and the subject is now supplemented by another on "Shear Force in T-Beams" given below.—ED.

In designing a T-beam the designer must take account of tensile and compressive forces and also of shear forces before deciding upon the section. These latter forces occur at more points in the section of a T-beam than in the section of a plain beam, and it is the object of this article to deduce expressions which will enable their values to be calculated.

Fig. I.



The elevation and end view of a T-beam of reinforced concrete are shown in *Fig.* 1, with the three principal sets of planes along which fracture by shear may take place.

Firstly, it may shear along the planes AB, A_1B_1, shown in the elevation, or any parallel plane. The shear along these planes, which are perpendicular to the axis of the beam, is usually considered in the design of beams.

Secondly, it may shear along the plane CD or some other parallel plane, such planes being all parallel to the neutral surface. This shear force has its maximum value at the neutral axis.

Thirdly, it may shear along the planes EF, E_1F_1; that is, the flanges may shear away from the web of the beam. This shear is a maximum at points where the flange is attached to the web, and decreases as the distance of the plane from the web increases. Let us now look closely into these shear forces and their cause or causes.

The shear force on any section is due to the loading upon the beam, and this loading also produces or causes tensile and compressive forces across planes which are normal to the neutral surface. In fact, all the forces which occur in a beam are due to the loading, and we must therefore consider the loading first, and then the connection between loading and forces.

In dealing with shear forces it will be found convenient to get a connection between shearing-force and bending-moment, the bending-moment being expressed as a function of the loading, and on this account the bending-moment at any point of a beam will first be deduced and then applied to the investigation of shear in the beam.

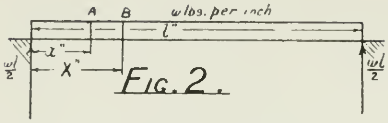


FIG. 2.

Two principal cases occur, viz.: a beam carrying a uniformly distributed load, and a beam with concentrated loads.

CASE I.—Beam with uniformly distributed load. *Fig.* 2 shows a beam of length loaded with w lb. per unit length. Taking all the units in lb. and inches, l will be in inches and w in lb. per in. run. Points A and B, distant x and X inches respectively from the left-hand end are taken, and the bending-moments at these points are found. They are :—

$$B_x = \frac{wlx}{2} - \frac{wx^2}{2} \text{ lb.-in.} \tag{1}$$

and

$$B_X = \frac{wlX}{2} - \frac{wX^2}{2} \text{ lb.-in.} \tag{2}$$

The increase in the bending-moment between the two points A and B will be given by the difference between (1) and (2), so that,

$$\text{Increase of bending-moment} = B_X - B_x = \frac{w}{2}\,[l(X-x)-(X^2-x^2)] \tag{3}$$

We now want to get a connection between the shear force at the points A and B, and the expression (3).

Shear force on planes perpendicular to the neutral axis, *i.e.*, planes AB and A_1B_1 of *Fig.* 1.

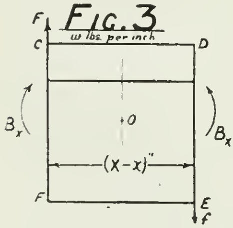


FIG. 3

Consider the external forces acting upon the piece $CDEF$ of a beam, *Fig.* 3, and keeping it in equilibrium.

The part of the beam to the left of the plane CF applies a shear force F upwards and a clockwise moment B_x as shown. The part to the right of the plane DE will apply the shear force f downwards and the counter-clockwise moment B_X, as shown.

In addition to the above there will be the vertically downwards force $w(X-x)$ lb., due to the distributed load on the top surface CD. As the section CF is nearer the support than the section DE, B_X will be greater than B_x and F greater than f.

Further, as the section CF is distant x, and DE is distant X, from the support, the values of B_x and B_X will be as given in (1) and (2).

Let us now take moments about O any point in the vertical line midway between CF and DE, we get

$$B_X - B_x = (F+f)\,\frac{(X-x)}{2} \tag{4}$$

Now the shear force diagram for a beam carrying a uniformly distributed load is a straight line inclined to the base, so that the shear force at O, midway between CF and DE, will be given by $\frac{F+f}{2}$, and calling this F_m, we may rewrite (4) as:

$$B_X - B_x = F_m(X-x) \qquad 5)$$

from which we get

$$F_m = \frac{B_X - B_x}{X-x} \qquad 6)$$

Substituting for $B_X - B_x$ from (3), we get

$$F_m = \frac{w}{2}[l-(X+x)] \qquad \text{(A)}$$

This gives the shear force on vertical planes which are distant $\frac{X+x}{2}$ from the support. In order to get a simpler expression we will take the planes CF and DE very close together, so close, in fact, that $X-x=O$, then $X=x$ and (A) becomes

$$F = \frac{w}{2}[l-2x] \qquad \text{(A}_1\text{)}$$

F being the total shear force at the point distant x from the end.

At the end where $x=O$, we get $F=\frac{wl}{2}$ lb.

,, centre ,, $x=\frac{l}{2}$, ,, $F=O$,

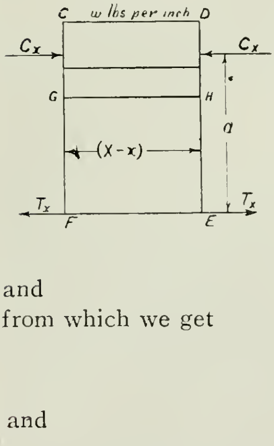
and so on for other values of x.

The area over which this shear force is distributed is $d \times b_r$, d being the effective depth of the beam and b_r the breadth of the web. Any assistance which the sides of the flange may contribute can only be very small, if any, and is therefore neglected.

The shear stress is therefore

$$f = \frac{w}{2db_r}[l-2x] \qquad \text{(A}_2\text{)}$$

Fig. 4.



Taking now the shear along planes parallel to the neutral surface, we replace the moments B_x and B_X by the total compressive forces C_x and C_X acting at the centres of compression of the concrete, and the tensile forces T_x and T_X acting at the centre of the steel reinforcement, as shown in *Fig.* 4. The lines of action of these forces are separated from one another by a distance equal to the lever arm a, and are connected with the bending-moments by the expressions:

$$\left.\begin{aligned} B_x &= aC_x = aT_x, \\ \text{and} \quad B_X &= aC_X = aT_X \end{aligned}\right\} \qquad (7)$$

from which we get

$$\left.\begin{aligned} C_x &= T_x = \frac{B_x}{a}, \\ \text{and} \quad C_X &= T_X = \frac{B_X}{a} \end{aligned}\right\} \qquad (8)$$

The difference between these latter expressions gives:

$$T_X - T_x = C_X - C_x = \frac{1}{a}(B_X - B_x) \qquad (9)$$

Now the piece $EFGH$ (*Fig.* 4) tends to slide to the right under the action of the tensions T_X and T_x, and is only prevented by the shear resistance along the plane GH. Of course, we may look upon the piece $CDHG$ as tending to slide to the left under the action of the compressions C_X and C_x, and being resisted by the shear resistance along GH. These are the only forces producing shear along planes parallel to the neutral surface, so that we get, taking F_1 as this shear force to distinguish it from the shear force on vertical planes:

$$F_1 = T_X - T_x = C_X - C_x,$$

and substituting from (9),

$$F_1 = \frac{1}{a}(B_X - B_x) \qquad (10)$$

Substituting now from (3), this becomes:

$$F_1 = \frac{w}{2a}[l(X - x) - (X^2 - x^2)] \qquad (\text{B})$$

This is the total force which acts over a length of $X - x$ inches. It will be convenient to consider its value for unit length, and this will be obtained by dividing (B) by $X - x$, thus:

$$F_1 \text{ per inch of length } = \frac{w}{2a}[l - (X + x)]$$

To make this comparable with expression (A_1) we must again put $X - x = O$, so that $X + x = 2x$, the latter x being the mean of the two former. Then the expression becomes:

$$F_1 \text{ per inch of length } = \frac{w}{2a}[l - 2x] \qquad (\text{B}_1)$$

At the end where $x = o$, we get $F_1 = \frac{wl}{2a}$ lb.

,, centre ,, $x = \frac{l}{2}$, ,, $F_1 = O$,

and so on for other values of x.

The intensity of shear force will be given by dividing (B_1) by b_r the breadth of the rib, thus:

$$f_1 = \frac{w}{2ab_r}[l - 2x] \qquad (\text{B}_2).$$

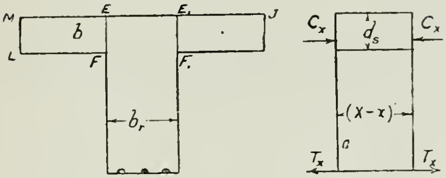
In the ordinary theory of beams it is stated that this shear is a maximum at the neutral surface and decreases as the plane considered gets further and further from the neutral surface. In the case of a homogeneous material such a statement is true, but in the case of a heterogeneous material, such as that we are now dealing with, it is not true, and we must therefore look to our assumptions before we can say anything about the variation of shear force on horizontal planes throughout the web. The two assumptions in the design of **T** beams with which we are now concerned are:

(1) The steel takes all the tension.

(2) The flange takes all the compression.

It follows at once, from these assumptions, that the horizontal sliding force is the same at all points of the web between the steel and the lower surface of the flange. Thus the tendency to shear the web along horizontal planes is the same at all points between the steel and the lower surface of the flange. As a matter of fact this statement will not be quite true because, no matter what our assumption may be, we cannot prevent the part of the web above the neutral axis from contributing to the compression C_x; the difference is, however, only on the side of safety.

FIG. 5.



To get the shear force tending to shear the flanges away from the web across the planes EF and E_1F_1, *Fig.* 5, we will consider the compressions C_x and C_X already found.

The piece $EFLM$ is subjected to part of the compression C_x which acts from left to right and part of the compression C_X which acts from right to left. It is assumed that these forces are distributed over the whole of the flange the breadth of which is b. The proportion of the whole which acts upon the part $EFLM$ will be the fraction $\frac{ME}{MJ}$.

Now $\frac{ME}{MJ} = \frac{\frac{b-b_r}{2}}{b} = \frac{1}{2}\left(1 - \frac{b_r}{b}\right)$, so that the total force acting on the area $EFLM$ from left to right will be $\frac{C_x}{2}\left(1 - \frac{b_r}{b}\right)$, and the opposing force will be $\frac{C_X}{2}\left(1 - \frac{b_r}{b}\right)$. The total sliding tendency acting towards the left-hand end of the beam will therefore be:

$$\text{Sliding force} = \tfrac{1}{2}\left(1 - \frac{b_r}{b}\right)(C_X - C_x) \qquad (11)$$

and substituting from (3) and (9) we get:

$$F_2 = \frac{w}{2a}\left(1 - \frac{b_r}{b}\right)\left[l(X-x) - (X^2 - x^2)\right] \qquad (c)$$

F_2 being the total shear force acting on a length $X - x$ of the flange on one side only.

We will now take this shear force per unit length of beam to make it comparable with F and F_1, and dividing (c) by $X - x$, we get:

$$F_2 \text{ per inch of length } = \frac{w}{4a}\left(1 - \frac{b_r}{b}\right)\left[l - (X + x)\right]$$

Putting $X + x = 2x$ as before, this becomes:

$$F_2 \text{ per inch of length } = \frac{w}{4a}\left(1 - \frac{b_r}{b}\right)\left[l - 2x\right] \qquad (c_1)$$

Other things being equal, this expression increases as the ratio $\frac{b_r}{b}$ decreases, and as the R.I.B.A. Report allows the limiting value $b = 6b_r$, the ratio $\frac{b_r}{b}$ is never, or should never be, less than $\frac{1}{6}$. Thus the maximum value of F_2, for any particular point of a beam, will be given by

$$F_{2\,\text{max.}} = \frac{5w}{24a}\left[l - 2x\right] \qquad (c_1{}^1)$$

At the end where $x = O$, this gives $F_2 = \frac{5wl}{24a}$ lb.

,, centre ,, $x = \frac{l}{2}$,, ,, $F_2 = O$,

and so on for other values of x.

This force acts over an area of $1\times d_s$, d_s being the thickness of the flange, so that dividing $(C_1{}^1)$ by d_s we get

$$f_2=\frac{5w}{24ad_s}[l-2x] \qquad (C_2)$$

We have already seen that the shear force, at each of the three sections considered, is a maximum at the support, and taking these maximum values by putting $x=O$ in the expressions (A_2), (B_2) and (C_2), we get

$$f=\frac{wl}{2db_r}=\frac{W}{2db_r}\text{ lb.} \qquad (A_3)$$

$$f_1=\frac{wl}{2ab_r}=\frac{W}{2ab_r}\text{ lb.} \qquad (B_3)$$

$$f_2=\frac{5wl}{24ad_s}=\frac{5W}{24ad_s}\text{ lb.} \qquad (C_3)$$

W being the total load on the beam in lb.

In the event of any of these being greater than the permissible shear stress, *i.e.*, 60 lb. per sq. in., the section must be reinforced by shear steel; the shear at other points must also be calculated to determine whether shear steel at these points is necessary.

EXAMPLE.

A **T**-beam has a flange (floor slab) of $4\frac{1}{2}$ in., a span of 20 ft., depth below the slab (including 2 in. of cover) of $13\frac{1}{2}$ in., breadth of web of 9 in., and carries a load of 1,800 lb. per ft. Given the lever arm as 14·11 in. find the maximum shear forces f, f_1, and f_2, and determine whether shear reinforcement is necessary. Here we have:

$W=1800\times 20=36{,}000$ lb. ($w=150$ lb. per in., $l=240$ in.).

$d=13\frac{1}{2}+4\frac{1}{2}-2=16$ in.

$b_r=9$ in.

$a=14{\cdot}1$ in.

so that we get

$$f=\frac{36000}{2\times 16\times 9}=125\text{ lb. per sq. in.}$$

$$f_1=\frac{36000}{2\times 14{\cdot}1\times 9}=142\text{ lb. per sq. in.}$$

$$f_2=\frac{5\times 36000}{24\times 14{\cdot}1\times 4{\cdot}5}=118\text{ lb. per sq. in.,}$$

and shear reinforcement across the three planes AB, CD and EF of *Fig.* 1 will be required at the end of the beam.

To get the point or points at which the safe shear resistance of the concrete just balances the shear force, we put expressions (A_2), (B_2) and (C_2) equal to 60 and find the values of x. In the case of the beam considered these will be found as follows:

$$60=\frac{150}{2\times 16\times 9}[240-2x] \qquad (A_2{}^1)$$

$$x=\frac{240-115{\cdot}2}{2}\text{ inches}=5{\cdot}2\text{ feet from the support for vertical planes in the web.}$$

$$60=\frac{150}{2\times 14{\cdot}1\times 9}[240-2x] \qquad (\text{B}_2{}^1)$$

$$x=\frac{240-101{\cdot}6}{2}\text{ inches}=5{\cdot}76\text{ feet from the support for horizontal planes in the web.}$$

$$60=\frac{5\times 150}{24\times 14{\cdot}1\times 4{\cdot}5}[240-2x] \qquad (\text{C}_2{}^1)$$

$$x=\frac{240-122}{2}\text{ inches}=4{\cdot}9\text{ feet from the support for vertical planes in the flange.}$$

It will be noticed that these are three different points.

It must not be forgotten that in the foregoing we have taken the maximum value of $\frac{b_r}{b}$, so that the calculated values referring to shear in the flange if they err at all only do so on the side of safety.

Case II.—Beam with Concentrated Loads.

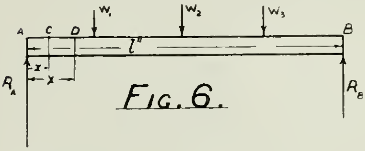


Fig. 6.

A beam with concentrated loads may carry only one, or it may carry more than one, and we will therefore consider the general case of a beam AB, *Fig.* 6, carrying loads W_1, W_2, W_3, &c.

The shear forces which exist between the support and the nearest load W_1 can be represented by one set of equations, while another set of equations will be required to represent the shear forces for the part of the beam between W_1 and W_2, and so on. The shear forces have their maximum values at the parts of the beam between the supports and the nearest loads, and decrease as the part of the beam between adjacent loads get nearer and nearer the centre of the span. On this account we will first consider the length of beam between the support and W_1, and then the length between W_1 and W_2, and assume that beyond this point the concrete is sufficiently strong without shear reinforcement.

Firstly for the length between the support and W_1.

All units in lb. and in. as before.

R_A is the reaction at A, and R_B that at B.

Taking now two points C and D distant x and X, respectively, from A, we get the bending-moments:

$$B_x=R_Ax$$
$$B_X=R_AX,$$

so that the increase of bending-moment in the length $X-x$ is

$$B_X-B_x=R_A(X-x) \qquad (12)$$

and re-writing (6) to apply to this case we get:

$$F_m=\frac{R_A(X-x}{(X-x)}=R_A \qquad (\text{D})$$

This expression tells us that the shear force on vertical planes in the web is independent of the distance of the point considered from the support, and writing F for F_m this force becomes:

$$F=R_A \qquad (\text{D}_1)$$

This acts over the area $d \times b_r$ so that the intensity of shear force is

$$f = \frac{R_A}{d b_r} \qquad (\text{D}_2)$$

If we wished to consider the shear at the right-hand end of the beam it would be necessary to substitute R_B for R_A in (D_1) and (D_2).

Equation (9) was deduced without reference to any particular form of loading, so that it still holds (see *Fig.* 4), thus the sliding force on horizontal planes will be:

$$F_1 = T_X - T_x = C_X - C_x = \frac{1}{a}(B_X - B_x),$$

and substituting from (12) we get

$$F_1 = \frac{R_A}{a}(X - x) \qquad (\text{E})$$

This is the total sliding force on a length of $X - x$, and dividing (E) by this we get

$$F_1 \text{ per inch of length} = \frac{R_A}{a} \qquad (\text{E}_1)$$

This acts over an area of $1 \times b_r$, so that the corresponding force per unit area is

$$f_1 = \frac{R_A}{a b_r} \qquad (\text{E}_2)$$

The expressions (E_1) and (E_2) are independent of the position of the point, so that the forces given are constant throughout the first length.

To get the shear across the flanges we proceed as in Case I and get the total sliding tendency on one side of the flange, and as *Fig.* 5 still holds for this case we may re-write (11) thus:

$$\text{Sliding tendency on one side} = \tfrac{1}{2}\left(1 - \frac{b_r}{b}\right)(C_X - C_x) \qquad (13)$$

and substituting from (9 and 12), we get

$$F_2 = \tfrac{1}{2}\left(1 - \frac{b_r}{b}\right)\frac{R_A}{a}(X - x) \qquad (\text{F})$$

To get this per unit length divide by $(X - x)$, thus

$$F_2 \text{ per unit length} = \frac{R_A}{2a}\left(1 - \frac{b_r}{b}\right) \qquad (\text{F}_1)$$

Taking, as before, $\frac{1}{6}$ as the limiting value of the ratio $\frac{b_r}{b}$, we get

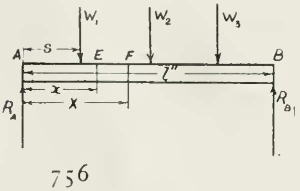
$$F_2 \text{ per unit length} = \frac{5R_A}{12a} \qquad (\text{F}_1{}^1)$$

acting upon an area of $1 \times d_s$, so that the intensity becomes:

$$f_2 = \frac{5R_A}{12 A d_s} \qquad (\text{F}_2)$$

and this is the same for all points between the support and W_1.

FIG. 7.



Secondly, for the length between the loads W_1 and W_2, see *Fig.* 7. The points E and F are taken distant, respectively, x and X from the end A. The bending-moments at these points are:

$$B_x = R_A x - W_1(x - s)$$
$$B_X = R_A X - W_1(X - s),$$

s being the distance of W_1 from A.

The increase of bending-moment in the length $X-x$ then becomes

$$B_X - B_x = (R_A - W_1)(X - x) \qquad (14)$$

Comparing this with (12), we may write

$$F_m = (R_A - W_1) \qquad \text{(G)}$$

and putting $F = F_m$ as before:

$$F = (R_A - W_1) \qquad (\text{G}_1)$$

which is constant throughout the length between the points where W_1 and W_2 are applied.

Dividing (G$_1$) by the area $d \times b_r$, the force per unit area becomes:

$$f = \frac{R_A - W_1}{d b_r} \qquad (\text{G}_2)$$

For the shear along horizontal planes in the web we get from (9)

$$F_1 = C_X - C_x = \frac{1}{a}(B_X - B_x) \qquad (15)$$

and substituting from (14) this becomes

$$F_1 = \frac{1}{a}(R_A - W_1)(X - x) \qquad \text{(H)}$$

Dividing (H) by $X-x$ to get the force per unit length,

$$F_1 \text{ per unit length} = \frac{1}{a}(R_A - W_1) \qquad (\text{H}_1)$$

and dividing this by the area $1 \times b_r$ over which it acts, the force per unit area is:

$$f_1 = \frac{1}{a b_r}(R_A - W_1) \qquad (\text{H}_2)$$

The forces given in (H$_1$) and (H$_2$) are the same for all points between W_1 and W_2.

The force tending to shear the flanges from the web will be given by writing $(R_A - W_1)$ instead of R_A in (F), thus:

$$F_2 = \frac{1}{2a}\left(1 - \frac{b_r}{b}\right)(R_A - W_1)(X - x) \qquad \text{(J)}$$

and the value of this per unit length is:

$$F_2 \text{ per unit length} = \frac{1}{2a}\left(1 - \frac{b_r}{b}\right) R_A - W_1) \qquad (\text{J}_1)$$

Taking now $\frac{1}{6}$ as the limiting value of $\frac{b_r}{b}$, the maximum values of (J$_1$) are given by:

$$F_2 \text{ per unit length} = \frac{5}{12a}(R_A - W_1) \qquad (\text{J}_1{}^1)$$

acting upon the area $1 \times d_s$, so that the intensity becomes:

$$f_2 = \frac{5}{12 a d_s} R_A - W_1) \qquad (\text{J}_2)$$

These forces are constant between the points where W_1 and W_2 are applied:

Collecting now the expressions giving the intensities of the shearing force of the various sections we get:

For the length between the support and the nearest force,

$$f=\frac{R_A}{db_r} \qquad (\mathrm{D}_2)$$

$$f_1=\frac{R_A}{ab_r} \qquad (\mathrm{E}_2)$$

$$f_2=\frac{5R_A}{12ad_s} \qquad (\mathrm{F}_2)$$

For the length between the load nearest the support and the next load,

$$f=\frac{R_A-W_1}{db_r} \qquad (\mathrm{G}_2)$$

$$f_1=\frac{R_A-W_1}{ab_r} \qquad (\mathrm{H}_2)$$

$$f_2=\frac{5(R_A-W_1)}{12ad_s} \qquad (\mathrm{J}_2)$$

An inspection of these expressions brings out two important points, viz.:

The intensity of shear force is the same at all points of a beam, carrying concentrated loads, between any two adjacent loads, the supporting forces being regarded as loads.

The intensity of shear force at any section of such a beam is a maximum in the end section, and is less the further the section is from the support.

In the event of any of the values given in (D_2) to (J_2) exceeding 60, the corresponding section must have shear reinforcement.

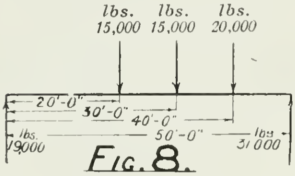


Fig. 8.

Example.

A T-beam of 50 ft. span carries loads of 15,000, 15,000, and 20,000 lb. at distances of 20, 30, and 40 ft. from the left-hand end respectively, as shown in *Fig.* 8. The slab or flange is 6 in. thick, depth of web below the slab, including 2 in. of cover, is 26 in., and the breadth of web is 14 in. Given that the lever arm is 27·5 in., find the intensity of shear forces f, f_1 and f_2, and say whether shear reinforcement is necessary.

Working from the left-hand end, we get:

$$R_A=19{,}000 \text{ lb.} \qquad W_1=15{,}000 \text{ lb.}$$

$$d=26+6-2=30 \text{ in.}$$

$$a=27{\cdot}5 \text{ in.}$$

$$b_r=14 \text{ in.}$$

$$d_s=6 \text{ in.}$$

For the section between the left-hand support and the nearest 15,000 lb. load:

$$f=\frac{19000}{30\times 14}=45{\cdot}2 \text{ lb. per sq. in.}$$

$$f_1=\frac{19000}{27{\cdot}5\times 14}=49{\cdot}3 \text{ lb. per sq. in.}$$

$$f_2=\frac{5\times 19000}{12\times 27{\cdot}5\times 6}=48 \text{ lb. per sq. in.}$$

As these intensities fall below the maximum permissible in the concrete—*i.e.*, 60 lb. per sq. in.—no shear reinforcement is necessary; further, it is not necessary to make any calculations for the middle section, as these will be smaller than above.

Working now from the right-hand end, we get

$$R_A = 31{,}000 \text{ lb.}, \quad W_1 = 20{,}000 \text{ lb.}$$

d, a, b_r and d_s as before, then for the length between the 20,000 lb. load and the support:

$$f = \frac{31000}{30 \times 14} = 73{\cdot}8 \text{ lb. per sq. in.}$$

$$f_1 = \frac{31000}{27{\cdot}5 \times 14} = 80{\cdot}5 \text{ lb. per sq. in.}$$

$$f_2 = \frac{5 \times 31000}{12 \times 27{\cdot}5 \times 6} = 78{\cdot}3 \text{ lb. per sq. in.}$$

Each of the above values is greater than the maximum permissible shear stress in the concrete, so that shear reinforcement will be required, across the three planes considered, throughout the whole of the end length of the beam.

For the part between the 20,000 and 15,000 lb. loads we get

$$R_A - W_1 = 31{,}000 - 20{,}000 = 11{,}000 \text{ lb.}$$

As this is less than $R_A = 19{,}000$ lb. at the left-hand end of the beam, and as the previous intensities obtained by using $R_A = 19{,}000$ lb. were less than 60 lb. per sq. in., then the intensities in this case will be still less, and need not be further considered. We had already decided that no shear reinforcement was necessary in the middle section of the beam when working from the left-hand end; but it was necessary to consider it when working from the right-hand end, because the intensities in the right-hand end section being greater than 60, left us no alternative.

Thus taking the beam as a whole, shear reinforcement will be required only in the right-hand section between the 20,000 lb. load and the support, and each of the three sets of planes, along which shear may take place, must be so strengthened.

THE CEMENT-GUN.

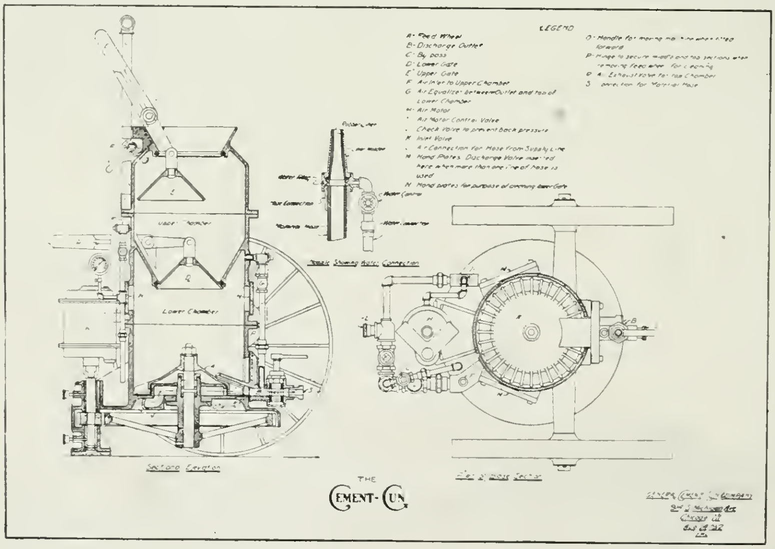
We give below some further particulars of the new method of applying cement mortar by means of the "Cement-Gun," and we think this mode of procedure is worthy of close attention by our readers, as it has already been used with great success in America.—ED.

In our issue of May, 1912, we published a paper describing the "Cement-Gun," which is a novel method of applying cement mortar by compressed air, and which has been used with great success in various large engineering projects in America, among others on the Panama Canal, where it has been employed to coat the disintegrating rock of the Culebra cut with concrete.

The apparatus previously described by us has been considerably altered and in its improved form is now being introduced into this country.

It is claimed that by this process a denser, stronger, more homogeneous and waterproof product is possible than by any other method. The term "Gunite" has been applied to this cement mortar to distinguish it from the ordinary hand-worked stucco.

In making "Gunite" the dry materials are forced through a hose in intermittent charges by means of compressed air, are hydrated at the nozzle, and applied to the work with a velocity corresponding to a pressure head of 35 lb. The combination of cement, sand, and water necessary to produce a plastic material is begun at the nozzle of the gun, and is completed during the instant of transit from the nozzle to the

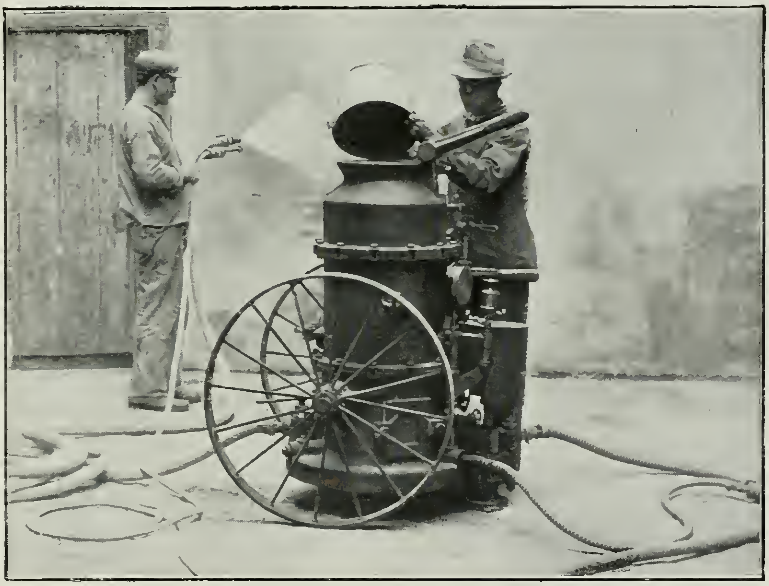

LEGEND
A - Feed Wheel
B - Discharge Outlet
C - By pass
D - Lower Gate
E - Upper Gate
F - Air Inlet to Upper Chamber
G - Air Equalizer between Outlet and top of Lower Chamber
H - Air Motor
Air Motor Control Valve
Check Valve to prevent back pressure
K - Inlet Valve
Air Connection for Hose from Supply Line
M - Hand Plates. Discharge Valve inserted here when more than one line of hose is used
N - Hand plates for purpose of opening Lower Gate
O - Handle for moving machine when lifted forward
P - Hinge to secure middle and top sections when removing feed wheel for cleaning
Air Exhaust Valve for top Chamber
S - Connection for Material Hose
Upper Chamber
Lower Chamber
Sectional Elevation
Detail Showing Water Connection
Water Control
THE CEMENT-GUN

Plan and Sections of the "Cement Gun."

work. It is known that the moment water is brought into contact with cement, the initial set or chemical combination begins. This initial set must be regarded as the first step in the hardening which gives the final strength to the mass. In this new method of applying mortar the initial set takes place on the work, and not on the mixing board. In applying " Gunite " only the necessary amount of water is used to secure the perfect hydration of the cement. This is obtained by the forcible application of the plastic to the work whereby all surplus water and air are effectually expelled.

" Gunite " is applied to any surface or to any material and of any shape, without the use of forms, and its surface may be left either in the rough or finished. It has been used for coating brick buildings, repairing sea-walls, building side-walks, and fortification work; coating the interior of tunnels and high-pressure syphons; also for coating heavy steel members at the New York Central and Hudson Railroad's improvements in New York City, and all the steel members of the new fifty-story Woolworth building in New York are being coated by this process.

The Cement-Gun was also used at the water department of San Francisco, California, where an interesting piece of work was accomplished which deserves some amplification. This work was in connection with a large oval reservoir designed to hold eleven million gallons of water for a high pressure fire system, and which was divided by a partition wall in the centre so that alternate sides could be drained for the purpose of cleaning. It was the intention of the constructors to fill one side first and have the invited guests stand in the empty side during the opening ceremony. However, on filling one side it was found, on account of the porosity of the partition wall, about 2 ft. of water covered the floor of the other side. A coating of " Gunite " was applied on each side of the wall, the entire work being accomplished in eight days. Immediately on the removal of the Cement-Gun outfit on the eighth day the water was turned on again in the one side, and the seepage of water was found to be absolutely stopped, and the dedication exercises were held as originally planned.



View of the Improved Pattern of the "Cement-Gun."

In connection with the use of "Gunite" for repairing sea-walls, we show below an illustration of its use for the Public Park Department of Lynn, Mass., U.S.A. The work was accomplished between tides in salt water and was completely successful. The tides had no effect whatever on the recently placed concrete.

Cement Gun at Work on the Sea Wall at Lynn, Mass., U.S.A.

As noticed in our issue of February, 1912, a series of tests were carried out by the well-known engineering firm of Westinghouse, Church, Kerr and Co., of New York, to determine whether concrete-surface coatings applied by this new air-blast device were equal or superior to hand-applied coatings. The report contained the following conclusions, which we publish again below:—

> "In all of the tests made the product of the Cement-Gun showed superior to good hand-made products of the same kind. The degree of superiority varied between wide limits.
>
> "In tensile strength the gun-work excelled hand-work in every case by amounts ranging from 20 per cent. to 260 per cent.
>
> "In compressive strength the excellence of the gun-work was even more marked, ranging from 20 per cent. to 720 per cent. better than hand-work.
>
> "In the matter of surface permeability the gun-work absorbed from $\frac{7}{10}$ down to $\frac{1}{20}$ as much water per hour per unit of area as the similar hand-made surfaces.
>
> "As regards absorption of water the hand-made mortars took up from 1·4 to 5·3 times as much as the gun-made mortars.
>
> "The percentage of voids of the gun-made product range from 22 per cent. to 87 per cent. of that of the hand-made product.
>
> "The adhesion of the gun-applied mortars was on an average 11 per cent. better than that of hand-work."

RECENT BRITISH PATENTS RELATING TO CONCRETE.

We propose to present at intervals particulars of British Patents issued in connection with concrete and reinforced concrete. These particulars have been prepared for this Journal by Mr. A. W. Farnsworth, of Strand Chambers, Derby.—ED.

Improvements in or relating to Moulds for Concrete Walls, Partitions, Floors, Ceilings, and the like.—*No.* 422/12. *Harold Williamson Lake. Accepted July* 11/12. —This is a highly-technical description from an American corporation of an invention for sectional moulds such as are employed in the construction of monolithic concrete buildings, walls, partitions, floors, ceilings, etc., the moulds consisting of a number of flanged plates fastened together edge to edge, and maintained the correct distances apart, for the insertion of the material forming the wall section by spacing pieces, and tie bars passing through perforations in the said plates. It is claimed that complications in providing for mould corners are much simplified, and that, by employing "mould-sections" at the corners of the mould which permit adjustment of the mould for various thicknesses of wall and for various odd lengths of wall while preserving registry of the apertures in the mould walls through which the transverse tie bars are to be passed, great gains result.

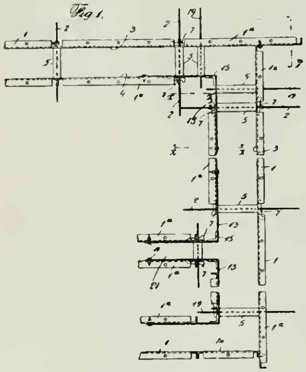


Fig. 1.

A considerable number of illustrations of different methods of erecting are given, and are minutely described. *Fig.* 1 will, however, give the practical builder an introduction of the lines upon which the specification is based. Ten different claims are made.

Improvements in and relating to the Construction of Reinforced Floors and Beams of Concrete or any other Fireproof or Fire-resisting Material.—*No.* 2762/12 *William John Swain. Accepted July* 4/12.—The inventor describes an improvement or modification of the ideas contained in his Specification No. 11,444 of 1911. This improvement consists in dispensing with the projecting flanges shown in the previous invention, as it is stated that experience has shown that they add nothing to the strength of the reinforced-concrete beams and joists, whilst at the same time it is necessary in that particular construction to arrange the lower members of the reinforcement at least an inch above said flanges in order to obtain the necessary strength of adhesion between the concrete and the reinforcing members. As this appears to be a wasteful method, the inventor shows in *Fig.* 1 a sectional view of a pair of blocks and an intersecting reinforced concrete beam, and states that the blocks (1) when placed in position for casting, beam *A* are supported by the planks (2) with the necessary number of props (3) to support the weight of the floor, until beams *A*, which are cast in the spaces between the blocks, have set and dried. Deeper beams, *A*, are formed by dropping planks (2), and supporting blocks (1) by side pieces. Serrations are formed in the ends of blocks (1) to key the beams to the blocks.

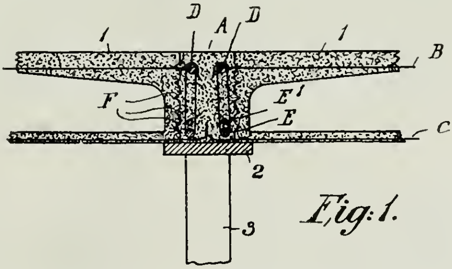


Fig: 1.

By adopting this method the inventor claims that the cost of the expensive wooden decking and centering required in reinforced concrete construction where the floors have to be cast *in situ* is, as will be seen by the drawing, saved by his method of construction.

Improvements in Concrete Water and like Tanks, and Methods of Manufacturing Same.—*No.* 6,458/12. *John Hugh McCoy. Accepted July* 11/12.—This is an improved method of manufacturing concrete water-tanks for carrying a body of water at a height above the ground such as is required for railway locomotive supply, etc., and is to provide a tank of such character, circular in cross section, having a lower base structure, and an upper tank portion thereon consisting of a floor or bottom, and a cylindrical wall extending above. The features of novelty claimed refer principally to the arrangement of the reinforcing elements and their embodiment with the main tank structure, and to the method of making the structure and reinforcing elements so as to provide a continuous integral body of concrete having the imbedded reinforcing elements.

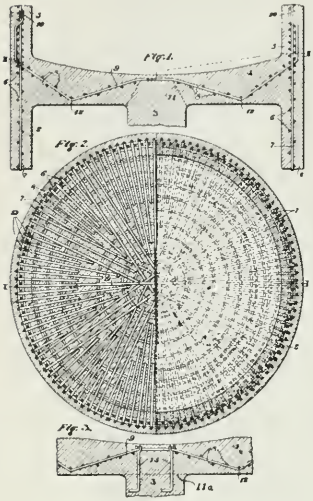


Fig. 1 will give a general idea of the inventor's proposals. It will be seen that the entire tank, including the sub-structure, is constructed of continuous integral concrete or cement. In order to withstand the different strains, the outer walls and the weight-supporting floor are strongly reinforced by imbedded tension rod members arranged in such a manner as to strongly tie the walls together through the floor, utilising the central supporting pier (3) as a middle rest for the transverse truss members which are firmly incorporated with the upper wall reinforcing members.

A long and technical description is given of the methods of building this tank, in which the inventor points out how he secures various advantages by his methods, and he provides for modified constructions in different ways. He points out that he avoids the use of H beams or other structural members, and that the continuity of the structure and its efficiency to withstand the strains of use result directly from the construction and results of the improved method of manufacture which he describes.

Improvements in or connected with Interlocking Concrete Slabs or Blocks for use in Building Concrete Walls, Sheet Piling, Groins and other Structures.—*No.* 6,980/12. *Robert Henry Neal. Accepted July* 25/12.—With the object of securely locking together a number of concrete slabs when used for building up groins, breakwaters, dams, partitions, walls, and other structures, the inventor provides them with overlapping or interlocking lugs which may be cast integrally with the slabs, or which may have a tongue and groove connection with adjacent slabs for duly locking together.

Fig 1

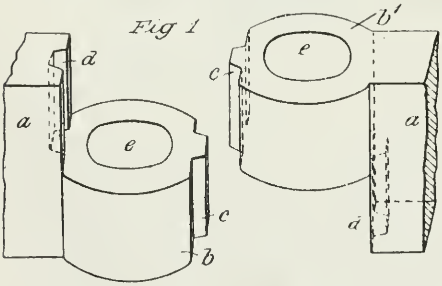
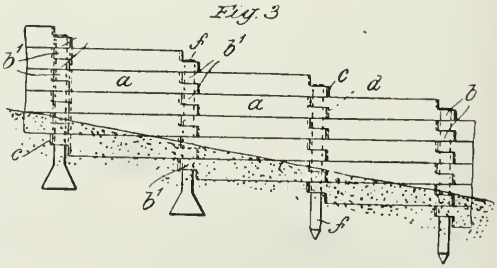


Fig. 3



The intention is that when the slabs are assembled together as shown in *Fig.* 3, concrete or other piles or fasteners may be driven through the lugs which have holes provided through them for that purpose. Thus the whole structure is rendered a solid body thoroughly tied together and into the ground.

Fig. 1 shows the lugs both as cast integrally and as cast loosely with tongue and groove connections.

Improvements relating to Reinforced Concrete Construction.—*No.* 15,929/11. *R. N. Mirza. Accepted July* 10/12.—In constructing reinforcement for concrete, the main stress member is preferably formed of an L-iron or other shape into the web of which is punched out straps having bow-like formation with locking notches. These straps are then fitted with rods of suitable section to act as continuity or shear members. In place of punching out the bow-shaped straps, the bow straps may be formed of other material adapted to be secured to the main stress bars. *Fig.* 2 shows an angle iron punched out at *A* to admit of fastening securely the shear member, *B*. *Fig.* 4 shows an alternative method of securing the shear member, *B*, to round bars, *D*, by means of a special shaped clip, *E*. Various other methods are described by which the same results may be obtained, but the two examples given here are characteristic of the whole.

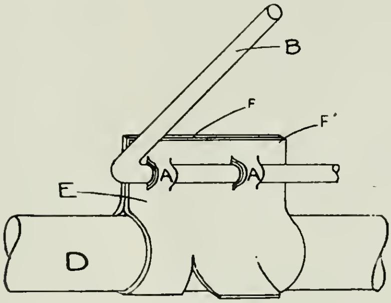


Fig 4

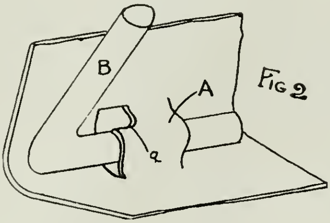
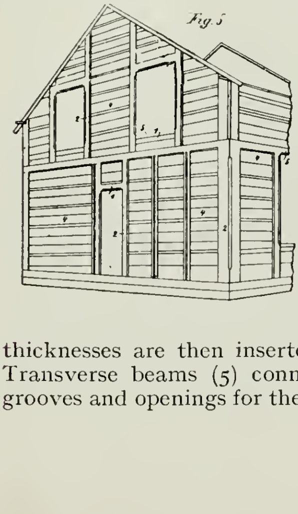
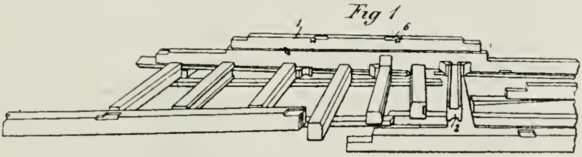


Fig 2

Process of Manufacturing Light Constructions of Reinforced Concrete.—*No.* 16,984/11. *Ernest Delille. Accepted July* 25/12.—Light buildings are made by the inventor by particular arrangement and combination of detail of sills, uprights, wall-forming slabs, and roof-forming slabs fitted together, the roof having a ridge extending from one end wall to the opposite wall. *Fig.* 1 is a persepctive view of the foundation sills and uprights which have just been taken out of the moulds and are ready for use. The finished sills (1) having indentations and grooves are placed in position, the uprights (2) are mounted upon them—these latter being provided with grooves and with window and door hinge fittings to engage recesses in the sills. Slabs of suitable thicknesses are then inserted in the grooves up to the upper ends of the uprights. Transverse beams (5) connect up the uprights, these beams having the necessary grooves and openings for the upper edges of the slabs, and for the heads of the uprights.





The roof is also made of slabs jointed by halved joints, and various other methods are described. *Fig.* 5 shows such a building in course of erection, and will give a good idea of the inventor's intentions.

Improvements in or relating to Concrete Building Construction.—*No.* 19,867/11. *Arthur William Rammage. Accepted August* 22/12.—This is an attempt to provide improved methods of building concrete walls and structures without the aid of the usual centering or boarding required by ordinary methods. It is claimed as advantages that the building may be plumbed in a much more convenient way, whilst it can be erected in a very economical manner, and, in the case of buildings in a country where transport is difficult and expensive, it reduces the quantity of material it is necessary to transport for erecting the structure. After referring to various known methods, the inventor describes his own manner of erecting a skeleton framework consisting principally of rows of scantlings spaced apart for the thickness of the wall that is to be erected, the scantlings being disposed alternately on the inside and outside so as to be more accessible. To the double-skeleton frame thus formed is attached Expanded Metal or wire netting of any usual type, the concrete being filled in between the netting or Expanded Metal, so that the scantlings may be removed, leaving the Expanded Metal or wire netting embedded in the concrete or in its surface. The netting or Expanded Metal will preferably be of such small mesh that, although it will be embedded in the surface of the wall, the concrete or cement will remain as separate protrusions through the mesh. These protrusions furnish a key for receiving rough plaster casts, cement, or any other suitable finish. The filling in of the concrete can proceed a few feet at a time, the netting being added as required until the desired height is reached.

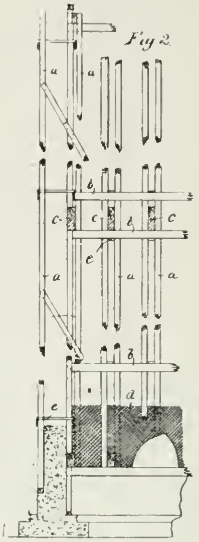


Fig 2.

The whole construction is then explained in detail, and a considerable number of claims made for it. *Fig.* 2 will show the vertical scantlings, *a a*, the method of attaching the netting or Expanded Metal *d* ready for filling up with the concrete as described. It is stated that the scantlings when placed outside the Expanded Metal will be generally removed, but can be allowed to remain if required, the whole being finished off by a facing in any suitable way.

Improvements in Helmets for Use in the Driving of Reinforced Concrete Piles.—*No.* 26,073/11. *Yorkshire Hennebique Contracting Co., Ltd.*, and *G. W. Crawshaw. Accepted July* 25/12.—The object of this invention is to provide an improved helmet for driving concrete piles. It is stated that the ordinary method of doing this involves the use of uprights having an over-all length of about 5 ft. which have to be kept in line with the piles by providing gauge bolts and "leaders," the whole arrangement being very heavy and cumbersome, and requiring much management and attention.

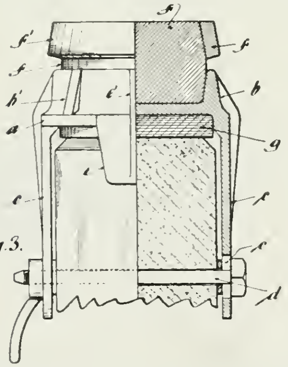


Fig. 3.

On reference to the drawing it will be seen that this helmet is very short, not exceeding a length of, say, a foot on the top of the pile, which enables it to be handled manually instead of by machinery, dispenses with the gauge bolt and also sawdust which has to be frequently renewed, and by shortening the helmet and "dolly" gives a better stroke to the pile driver, thus aiding operations materially.

Fig. 3 will give a good idea of the apparatus—*a b* is the body of the helmet, *c* being guiding arms prolonged down the pile with slots adapted to take a bolt, *d*, which passes through a hole in the top of the pile, and is used for_keeping the helmet vertical and in position ; *g* is a wood block interposed between the metal body and the top of the pile in place of the usual sawdust; and *f* is the "dolly" banded as at f^1. Various modifications are also described.

Improvements in Fire-proof Floors and Partitions Constructed in Reinforced Concrete.—*No.* 26,805/11. *G. E. Montagnon and F. Atkinson. Accepted June* 27/12. —This is an arrangement by which reinforcing of concrete beams for floors with the metal truss described in combination Specification No. 20,844 of 1909 is described. The arrangements provide for the floor and ceiling slabs to form a continuous or uninterrupted floor and ceiling surface. Reference to *Fig.* 1 will show that the beams (*A*), which may be cast either *in situ* or elsewhere, are reinforced at *F*, and are formed with a rebate at (*G*) on their upper portion, which may be continuous or interrupted as desired, in order to form a bearing for the floor slabs (*C*). The lower portions of the beams (*A*) are also provided with projecting fillets, or horizontal ledges (*H*), which form bearings for the ceiling slabs (*B*). A method of constructing the timber moulding for beams (*A*) is described in the specification, and consists briefly of a method such that the side timbers may be removed to facilitate the drying of the concrete, leaving the bottom timbering intact and supported from the metal truss (*F*), as described and illustrated in the previous patent No. 20,844/09. The object of this is that no undue stress shall be put on the beam during the process of drying out.

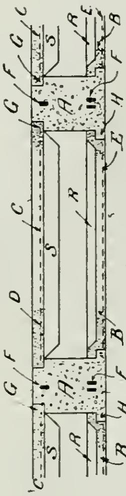


Fig. 1.

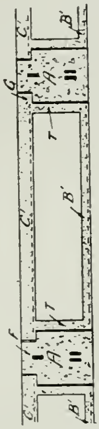
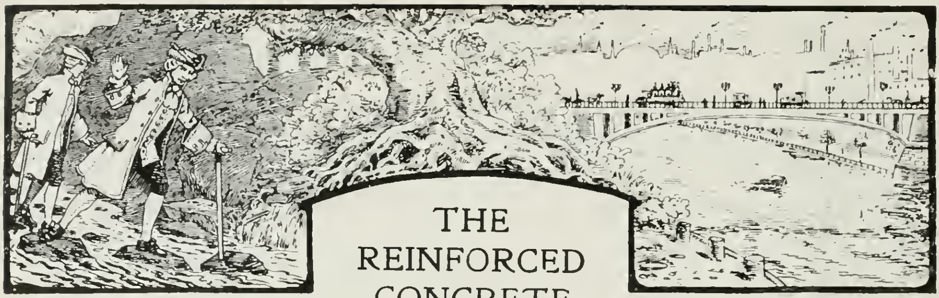


Fig. 4.

The flooring slabs (*C*) and the ceiling slabs (*B*) are, of course, provided with suitable reinforcements (*D* and *E*), and are cast separately or together and hoisted into position between the beams.

An alternative method of construction is also shown in *Fig.* 4, in which the flooring and ceiling slabs are cast together, being connected at the sides by means of ribs (*T*) and in this case the bottom projecting ledges (*H*) in *Fig.* 1 may be dispensed with. Various other alternative methods are also described, but the above give the general idea of the invention.

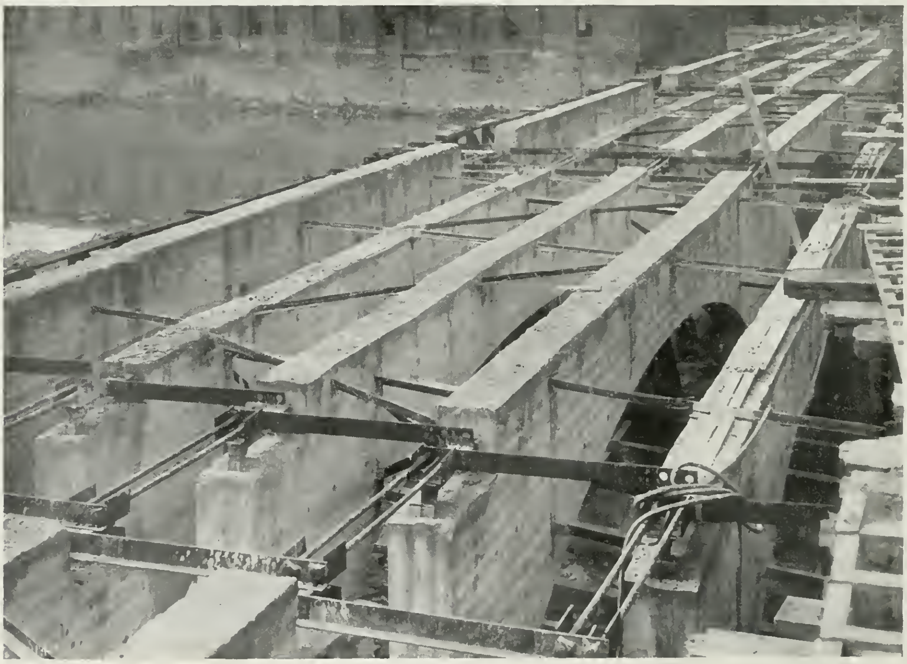


THE REINFORCED CONCRETE BRIDGE AT CENTRAL AVENUE, ROCHESTER, U.S.A.

A REINFORCED CONCRETE STRUCTURE COMBINED WITH A CONCRETE DAM AND HEAD-WORKS FOR POWER DEVELOPMENT.

The description given below of the construction of a reinforced concrete bridge and concrete dam combined is a very interesting one, our particulars having been taken from the "Engineering Record" The photographs were kindly put at our disposal by the Department of Engineering of the City of Rochester, U.S.A.—ED.

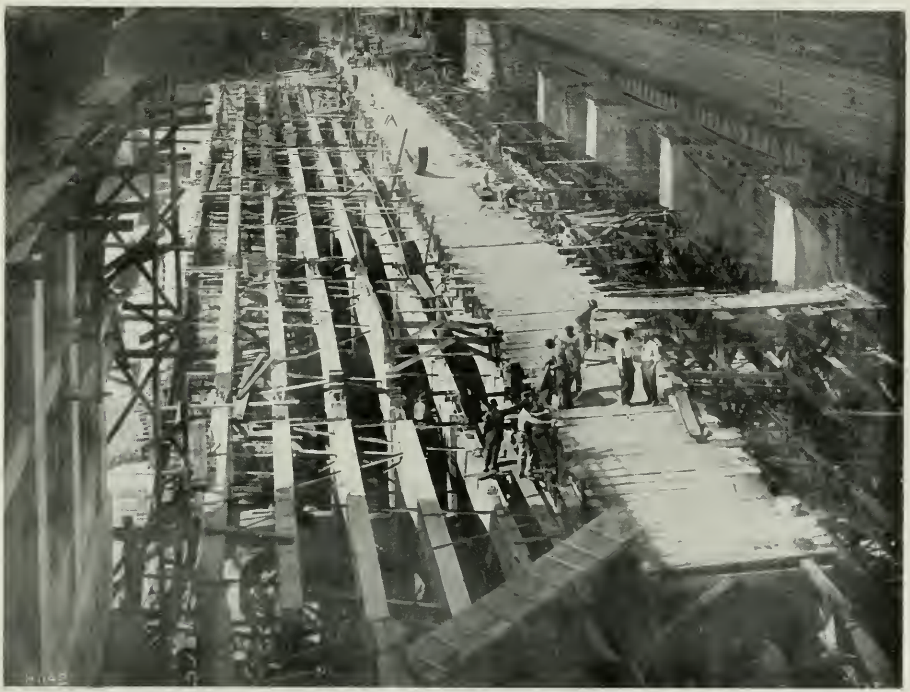
A REINFORCED concrete bridge made up of a series of nine arches has recently been built by the city of Rochester across the Genesee River at Central Avenue. At this point a portion of the waters of the stream are diverted for the development of power, and there were also included in the improvements the construc-



View of Concrete Arch Ribs.
REINFORCED CONCRETE BRIDGE AT CENTRAL AVENUE, ROCHESTER, U.S.A.

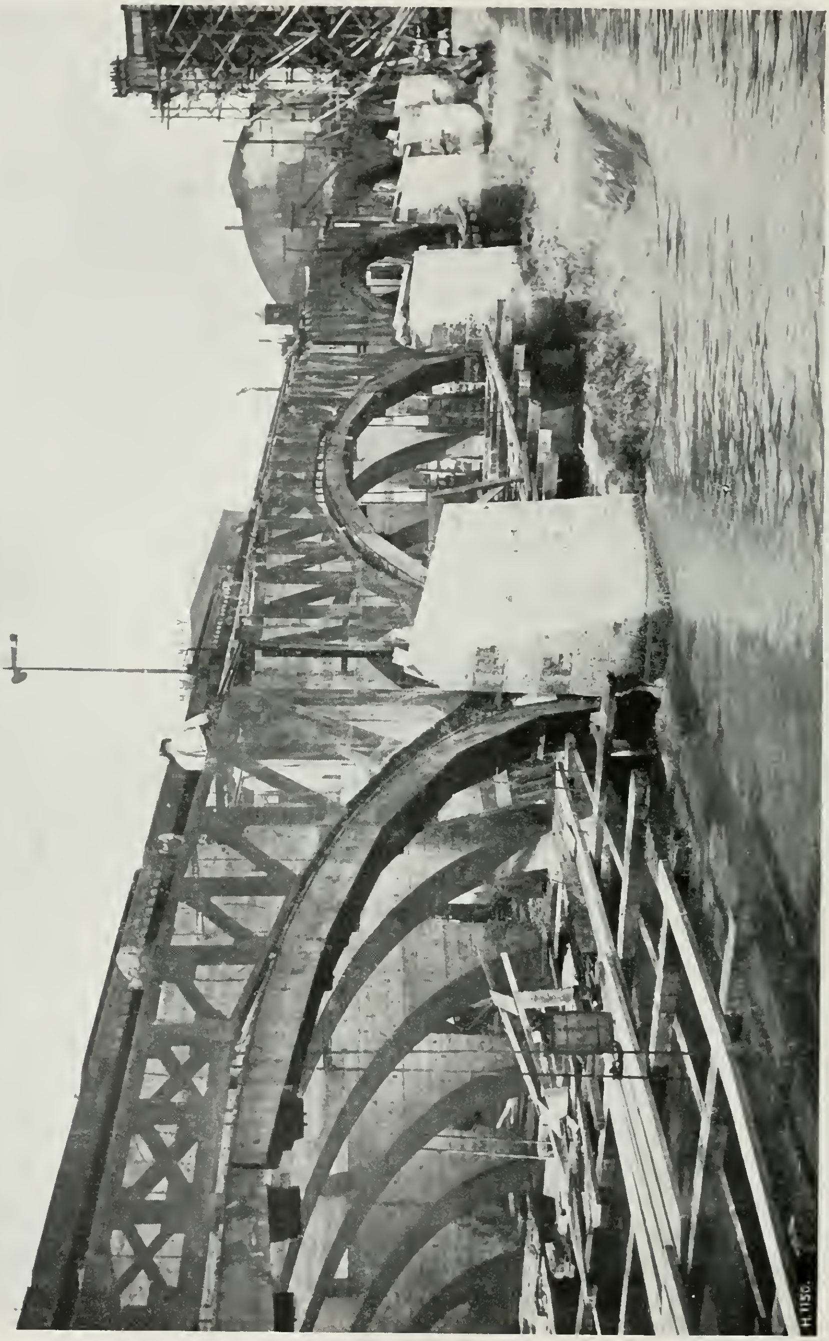
tion of a concrete dam over a certain section of the stream and concrete headworks for Brown's power race, which is located along the west bank. The new structure is 494 ft. long and 66 ft. wide over all. In the middle there is a depressed section 18 ft. wide and 14 in. deep, in which are laid the double tracks of the New York State Railways Company. On either side there is a 12-ft. roadway and beyond that two sidewalks each 10 ft. 10 in. wide.

The present bridge replaces a steel structure carried on three lines of trusses with a total width of 46 ft. 2 in. It was supported on nine piers and two abutments, and all but the east abutment were used in the support of the new bridge.



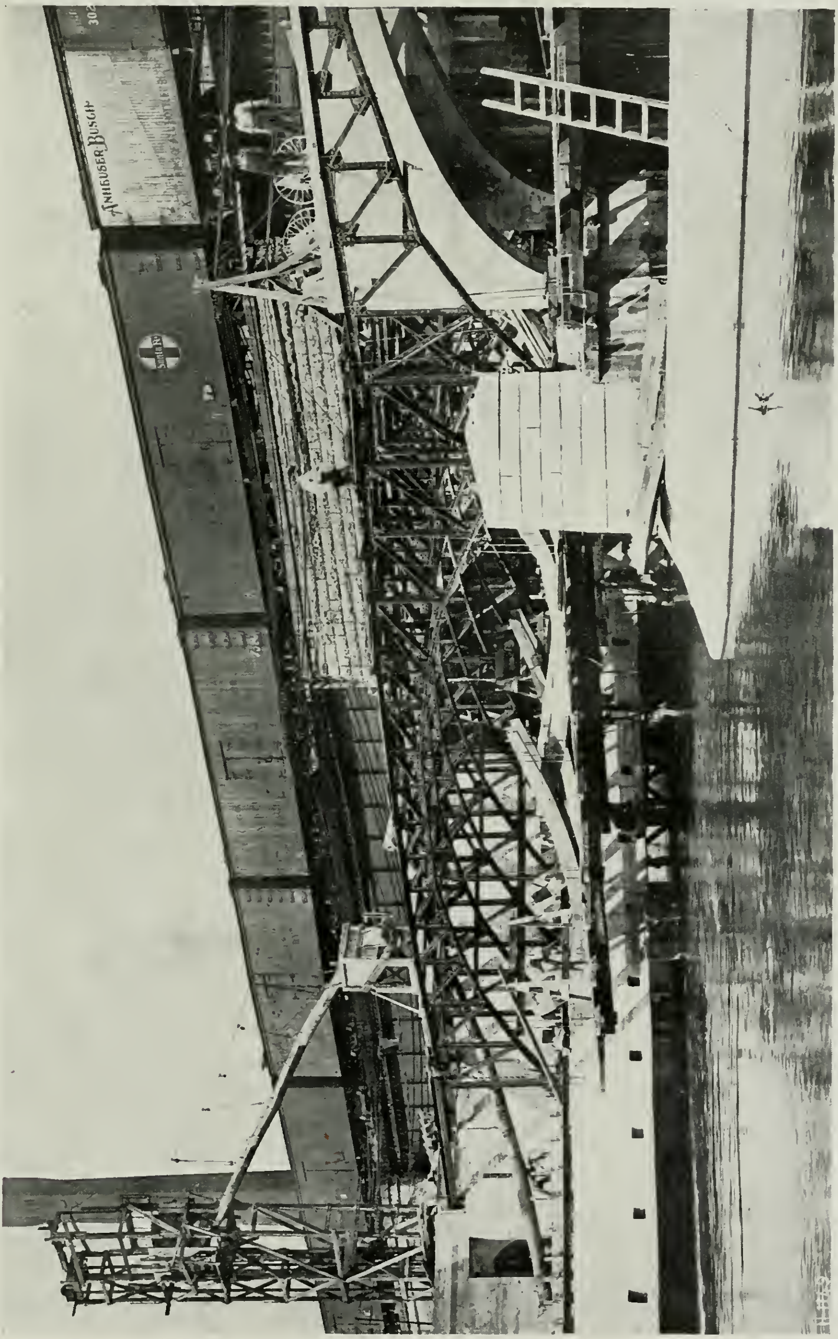
View of work in progress.
Reinforced Concrete Bridge at Central Avenue, Rochester, U.S.A.

At the east bank the old masonry abutment was torn out to rock and a new concrete abutment was built at a slightly greater distance from the first pier. This, it is stated, was done to allow for additional power development under consideration at present. From the east side the first eight piers are parallel to the east abutment and make an angle of 68 deg. and 30 min. with the centre line of the bridge. The west abutment, however, forming as it does the wall for Brown's race, makes an angle with the river wall opposite the end of pier No. 8 and diverges from the line of that pier, making an angle of 131 deg. 30 min. with the centre line of the bridge, this angle being measured in the same direction as the previous one. On account of the increasing span due to this divergence between the abutments and pier No. 8 an additional pier was built out from this one parallel to the face of the west abutment. At the east bank



Structural Reinforcement and Arch Rib Forms in place.
REINFORCED CONCRETE BRIDGE AT CENTRAL AVENUE, ROCHESTER, U.S.A.

the water is diverted to the power plant of the Rochester Railway and Light Company.



View of Bridge in course of construction.

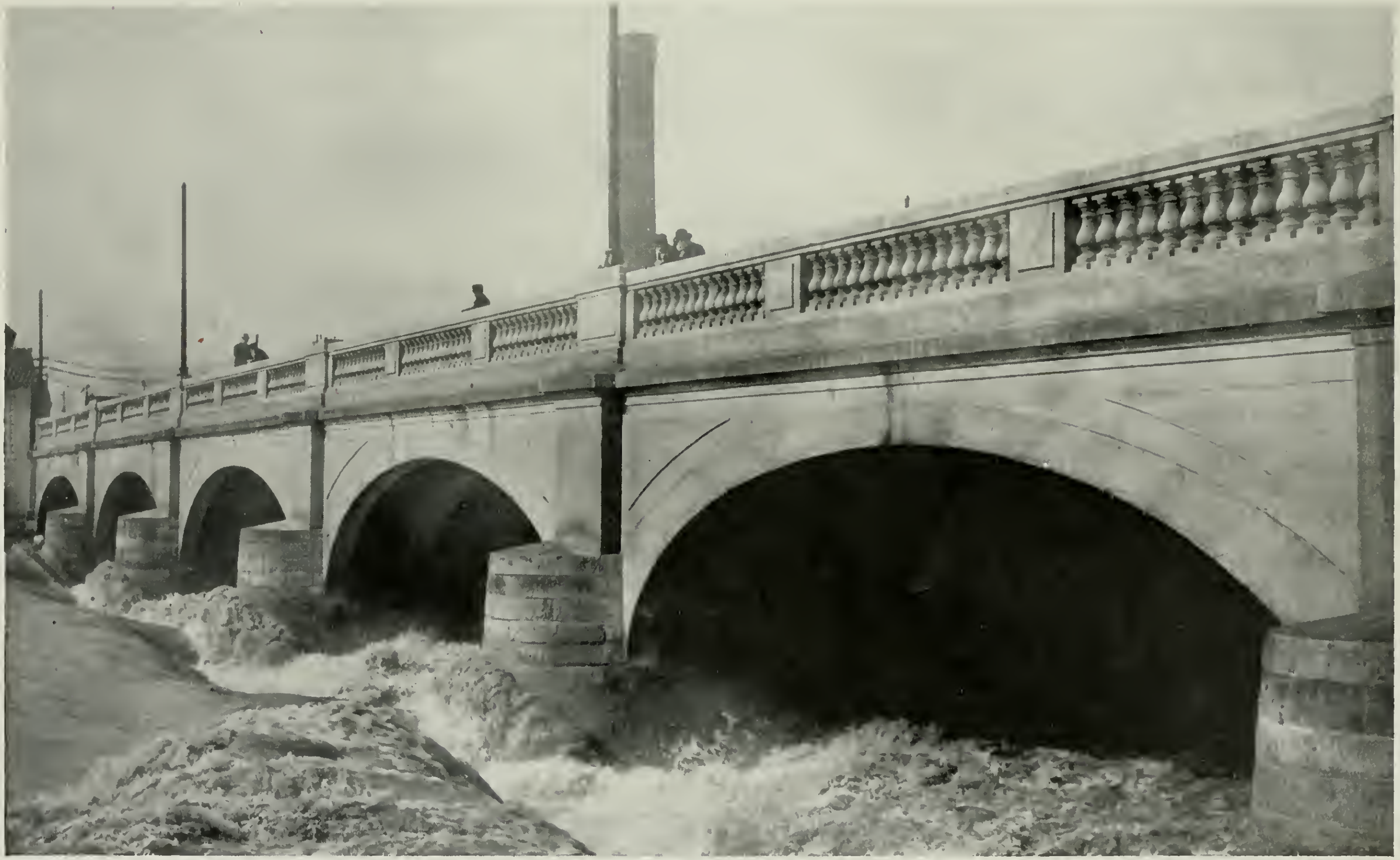
Reinforced Concrete Bridge at Central Avenue, Rochester, U.S.A.

On account of the increased width of the new structure it was necessary to build out concrete additions to the existing piers. To accomplish this the masonry nosings were removed from piers 3 to 7, inclusive, and the abutting walls from piers 1 and 2. The masonry of the piers was then removed to an elevation below the springing lines of the arch ribs and along the outer surface of the piers a sufficient amount to enable the bonding of a shell of concrete which was placed over the entire lateral surface. This shell was 12 in. thick and reinforced with ⅜-in. vertical and ¾-in. horizontal square twisted bars spaced 12 in. on centres, and the horizontal bars in the additional sections were bonded with the concrete shell of the old piers. The additional sections and the annular shells of concrete were anchored to the old piers by rods placed in holes drilled in the old masonry. The concrete addition is carried to a height of 3 ft. 6 in. above the springing line of the arch ribs and is made symmetrical, with a width of 7 ft. at the springing line. After placing this concrete, the semicircular granite nosings for the piers were replaced and anchored to the piers with 1-in. dowels.

The two abutments are buttressed retaining walls with buttresses 16 in. wide spaced 5 ft. on centres opposite the arch ribs of the bridge. Each is provided with a bridge seat 2 ft. 5 in. wide, furnishing a bearing for the ends of the arch ribs. In the east abutment, which is entirely of concrete, there are 2¾-in. by ½-in. anchor rods extending from the arch rib section down through the section of the buttress and anchored in the rock below the concrete base of the abutment. In the case of the west abutment use was partly made of the existing masonry, and the earth was removed from behind it to afford a means for placing the necessary buttresses at the back of the wall. The masonry in the front face of the abutment was cut down to the required elevation for the bridge seat and a concrete back wall and additional section was built above it. The method of reinforcing employed in the two abutments is essentially the same. The anchorage for each arch rib in the west abutment consists of two bars 2¼ by ½ in. in section and 65 ft. long. These bars are bent around the semicircular end of a 4-in. cast-iron saddle, which is securely anchored in the concrete. At the section where the concrete of the arch rib meets the concrete of the abutment a sliding joint is provided by wrapping the ends of these anchor rods with greased tar paper to allow the bridge to move westward when necessary without putting a compressive stress into these rods.

At the north end of the east abutment a concrete retaining wall is built, running east and west, which retains the driveway to the Rochester Railway and Light Company's power station. All foundation work, together with the concrete facings of the old masonry piers, was completed in the fall of 1910, the work being done while the old structural steel bridge was still in use. The new dam was also built at that time, and it is stated that the utmost care had to be exercised in all of the work not to divert or blockade either of the two power races on either side of the stream. For this reason at no time could any of the water be diverted, so that all foundation work had to be done by means of narrow coffer dams placed adjacent to the piers and abutments, which required additional concrete.

The superstructure comprises a series of arch ribs spaced 5 ft. on centres and tied together with transverse beams which carry the floor system of the



A view of the finished structure.

Reinforced Concrete Bridge at Central Avenue, Rochester, U.S.A bridge. These arch ribs are made up of a truss work of angles and plates securely riveted together and embedded in concrete. These ribs are 16 in. wide and support an 8-in. reinforced concrete slab in the roadway and beneath the car tracks and a 6-in. slab at either sidewalk. The arch ribs vary in span from about 50 ft. over the greater portion of the structure, with a rise of 9 ft. 6 in. to 76 ft. over a portion of Brown's race with a rise of 10 ft. 6 in.

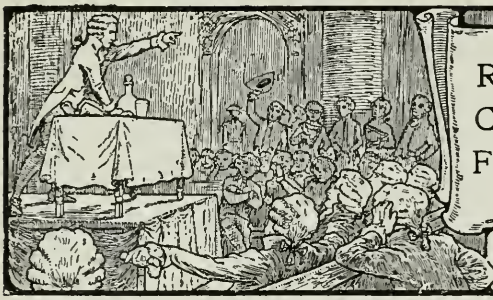
There is an ornamental parapet and balustrade of reinforced concrete along the outer edge of the bridge, which comprises a moulded base surmounted by large and small panelled posts, a line of balusters and a heavy reinforced handrail.

In connection with the extension of the bridge piers the concrete dam was likewise built. This dam extends between the faces of the third and seventh piers as measured from the east end and is joined to these piers by a curved section at either end. From the intermediate piers concrete buttresses are built to the dam section. These are 10 ft. long as measured from the base of the nosing of the piers to the inside of the crest of the dam and are 4 ft. wide and about 4 ft. high. The dam is built of mass concrete on a rock foundation and is anchored to this by a key 3 ft. wide and 2 ft. deep. It is built in sections about 10 ft. long with keyed sliding joints, the joints being covered with tar paper.

The gate-house for the head-works of Brown's race is a two-story reinforced building supported on reinforced concrete girders extending from the south end of the west abutment to an isolated pier built parallel to the abutment opposite the end of Pier No. 8 for this specific purpose. There is a 4-ft. walk extending from the abutment to the grating platform of the gate-house, giving access to the structure. The gates are operated in nine cast-iron sets of guides which are anchored in the concrete at the race bottom and likewise in the floor of the gate-house.

Special attention was given to the surfacing of the concrete in the bridge structure. All exposed surfaces, including the superstructure of the gate-house, were given granolithic surfaces, consisting of one part of white Medusa cement, two parts of clean white sand, and three parts of light-coloured granite not exceeding ¼ in. in size. They were likewise bush-hammered.

The principal concrete mixing plant was located at the west end of the bridge in the excavation behind the west abutment, about 22 ft. below street level. This enabled dumping directly from wagons into the supply hoppers of the material bins. A Milwaukee continuous mixer with a capacity of ¾ yd. was used at this point and the concreting materials were drawn directly from the supply bins into the automatic charging hopper of the mixer, while the concrete was elevated about 50 ft. above the bridge to a distributing hopper, from which it was transmitted by means of chutes to the desired location. The bulk of the concrete was handled from this end of the bridge in this way, but beyond the range of the chutes Ransome concrete carts were used on a centre runway built at such a height that the remaining portion of the structure could be reached without difficulty by means of auxiliary chutes. The bridge was designed by Mr. E. A. Fisher, city engineer, and the construction was handled by Mr. John G. Ellendt, engineer for Whitmore, Rauber & Vicinus, the general contractors on the work.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

ROYAL INSTITUTE OF PUBLIC HEALTH.

THE USE OF REINFORCED CONCRETE IN ARCHITECTURE.

Paper by HENRY ADAMS, M.Inst.C.E., M.I.Mech.E., etc.

The following is an abstract of a paper on the above subject which was read by Mr. Henry Adams at the Congress of the Royal Institute of Public Health, held in Berlin in July last.

THE constructive branch of architecture has in the last few years undergone a remarkable development through the introduction of two new methods of construction which have many useful characteristics. The first of these is that known as "steel frame construction." When the London Building Act of 1894 was promulgated, the use of brick or stone was alone contemplated, and rules were laid down for the safe construction of buildings in these materials. Rolled joists and other metal sections had been in use for some thirty-five years as subsidiary details, but they did not form an integral part of the structure. The exigencies of modern requirements for speed and economy, however, induced architects to adopt steel framework more and more for the interior of their buildings, combined with concrete in the floors for fire-resisting purposes. This enabled building operations to proceed more rapidly, as several floors could be in progress at the same time; but the restrictions of the Building Act prevented them from obtaining the full benefit of the method, as the external and party walls had to be carried up of the prescribed thicknesses whether the loads were carried by the interior construction or not. Architects saw that if they could apply the same principle to the external walls much greater speed would be obtained, and the period during which the capital expended on the site was lying dormant would be greatly curtailed, in consequence of which the London County Council were induced to promote their "General Powers Act" of 1909, in which special regulations were framed for the construction of steel-framed buildings. If the stability of the structure was provided for by the steelwork alone, the external walls need not be more than brick panels filled in between the framework with such architectural features added as the architect might desire in order to produce a proper effect. The Ritz Hotel, Piccadilly, was the first building of any magnitude constructed wholly upon this principle, and special permission had to be obtained for it, as the new rules were not then in force. The experience with this building showed the facilities which the method of construction gave for rapid progress, and it was clearly proved that it was the most desirable procedure where large buildings were concerned. The *Morning Post* building followed shortly after, and since then the Royal Automobile Club was erected in a similar manner, but in the latter building many new features were introduced in the use of reinforced concrete.

This brings us to the second modern development. None of the previous Acts or Regulations contemplated the construction of buildings wholly or chiefly of reinforced concrete; but, as Government works are outside any building regulations, Sir Henry

Tanner, the Architect of His Majesty's Office of Works, was able to design and carry out a large General Post Office on this system. The economy of construction was most marked; the buildings, of six and seven floors, with a total area of nearly ten acres, cost only about 2¼d. per cubic foot of total contents, excluding stone fronts.

Other architects wished to follow suit, and the London County Council are now drawing up regulations to cover its use. These regulations are at present only in draft; they have been subject to much criticism, but they are based upon suggestions of the Royal Institute of British Architects, the Concrete Institute, and other public bodies concerned in the use of this material, and there is no doubt that they will provide a simple and complete set of instructions which will enable a building to be designed with safety and economy. The regulations are at the present time undergoing their final revision before being sanctioned by the Local Government Board, and, although it is scarcely to be hoped that they will not meet with opposition on the part of the specialist firms who would like a free hand, all the objections that have so far been made public have been met in a conciliatory spirit and very little real hardship is likely to arise in the interpretation of the regulations by the district surveyors. In the case of steel-framed buildings, the District Surveyors' Association have drawn up outline forms showing the particulars and calculations that are required to accompany the plans before they can be examined and permission granted for building; and doubtless they will prepare similar forms dealing with reinforced concrete construction, so that the architect or engineer acting for the building owner may know exactly what he is required to furnish, and all applications will be made upon the same basis.

The general method of this form of construction is now well known, but it may be useful to go a little into detail with regard to some of the more prominent characteristics. It is, of course, understood that the main principle consists in such a combination of the two materials, steel and Portland cement-concrete, that the steel takes the tensile stresses and the concrete the compressive stresses, so that each is used to the best advantage and therefore most economically. In certain parts of a structure it may happen that the steel also takes compressive stresses, as in pillars and over the supports of continuous beams, but this is merely incidental. Floors constructed upon this method usually consist of main beams, cross beams and the floor slab so proportioned that the minimum of material produces the maximum result. The floor slab being continuous and only having to span 6 ft. to 10 ft. need only be 4 in. or 5 in. thick; this rests upon the cross beams, say, about 10 in. or 12 in. deep by 6 in. or 8 in. wide; but, the work being monolithic, part of the floor slab acts as a top flange to the beams and converts them into tee beams. These cross beams in turn rest upon the main beams, say, 24 in. to 28 in. deep by 10 in. or 12 in. wide. An angle fillet is formed at all the junctions instead of leaving a plain square junction, and an angle bracket is formed below all beams at their resting points. The steel reinforcement bars are provided to meet the maximum bending moments in the centre of the beams, and then some of them are turned up towards the ends to meet the shear stresses. In addition, there are steel stirrups provided on these rods, closer together at the ends of the beams and wider toward the centre, to take the general shear stress which increases from the centre towards the ends. Some of the early failures were due to lack of provision for shear. The floors are supported by reinforced concrete pillars and by the walls. The pillars consist of vertical steel rods embedded in the pillar 1½ in. from the outer surface, and hooped round with small rods to prevent them from bulging when the load comes on. The walls practically consist of pillars and panels constructed on the same general principles.

Reinforced concrete construction is particularly adapted for resisting the effect of a fire, provided that limestone has not been used in making the concrete. Coke breeze makes a fire-resisting concrete, but is deficient in strength and expands when water is applied. Hard broken brick, of varying size, passing a ¾-in. mesh and retained on a ¼-in. mesh, is perhaps the best material for the larger aggregate, and clean sand passing a ¼-in. mesh and retained on a 1-50 in. mesh the best for the smaller aggregate, the mixture being 1 part of standard Portland cement, 2 parts sand, and 4 parts broken brick.

Thin partition walls are easily made in reinforced concrete with a total thickness of 1½ in. or 2 in. The reinforcement may be wire netting, Expanded Metal, wire lattice, or dovetailed corrugated steel, erected in place and coated on both sides.

Although mention has only been made of the principal features, it is usual in this kind of construction to adapt the material to all the various purposes, including roofs, stairs, retaining walls, tanks, etc. Roofs may be arched, or pitched, or flat. Arched roofs are designed as flexible arches for the reason that they will withstand tension. Pitched roofs are constructed usually upon the collar beam principle for ordinary buildings, or of the saw-tooth or weaving-shed pattern, with north lights, for a warehouse or factory. Flat roofs are constructed in the same way as floor slabs, but with falls to the rain-water gutters. Staircases are made in various ways, but generally with a reinforced beam for the outer string and the treads and risers as floor slabs.

Many endeavours have been made to render concrete waterproof. The earlier systems included the insertion of a layer of asphalte, then the insertion of flexible sheets containing indiarubber in their composition, but latterly the concrete itself has been relied upon with certain modifications. At the Farlington Reservoirs of the Portsmouth Water Co. the floors and walls were covered with two coats of cement rendering, the first coat being ¾ in. thick and the finishing coat ¼ in. thick. In another case of a reinforced concrete wall subject to inundations, an outer coating of 1½ in. cement mortar prepared with fine river sand and a solution of soft soap in water (4 lb. soap per cubic yard of concrete) was laid on, and this again covered with 3½ in. of fine-grained concrete. By another method, powdered alum is mixed with dry cement and sand, and hard soap is dissolved in the water used for gauging the concrete. The alum and soap combine and form a finely divided flocculent, insoluble, water-repelling compound which fills the pores of the concrete. The usual proportions are 1 part alum and 2 to 3 parts hard soap to 100 parts of water, the water used being about 20 per cent. of the bulk of the concrete. Sometimes ordinary dry hydrated lime is added to the cement, in the proportion of 10 lb. lime to each 1 cwt. of cement, the sand and larger aggregate being carefully graded and the mixture made with a full supply of water. Other constructors use the lime in a rendering mixture only, two coats each ½ in. thick of 10 per cent. lime, 90 per cent. cement, and 200 per cent. sand, the walls being first brushed and cleaned with steel brushes. No settled practice yet exists, and it is very desirable that some public body should undertake an exhaustive series of experiments to ascertain the simplest and best means to employ to ensure water-tightness.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

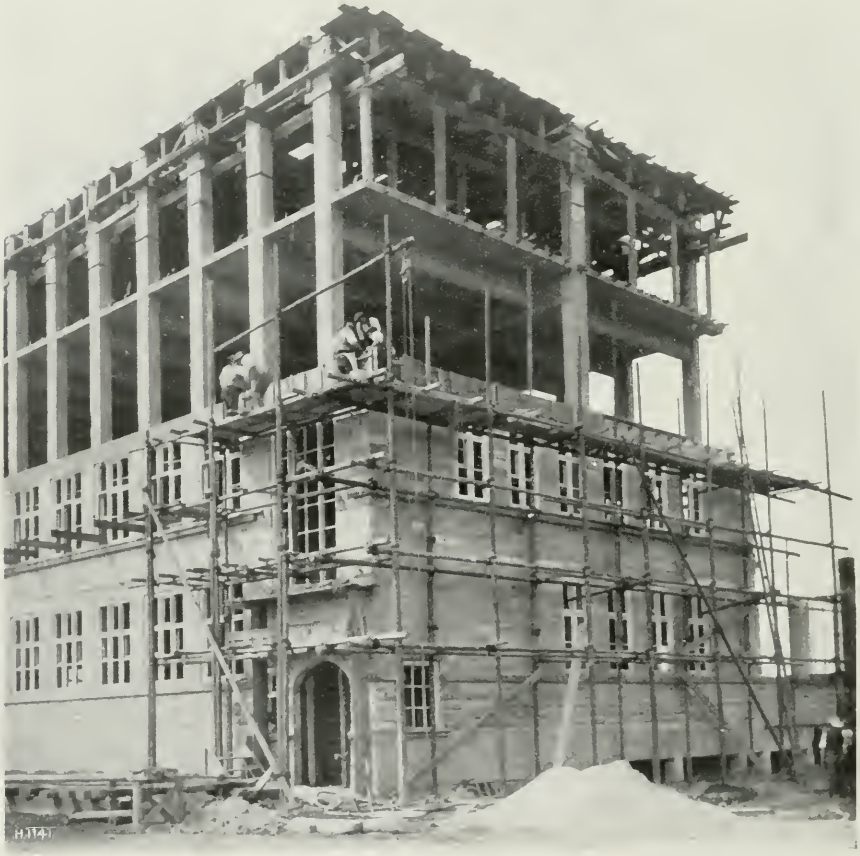
Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

REINFORCED CONCRETE FACTORY FOR THE UNION CABLE COMPANY.

A LARGE reinforced concrete factory has recently been completed for the Union Cable Company at Dagenham Dock, Essex, by Messrs. Samuel Williams and Sons, Ltd., of Dagenham Dock.

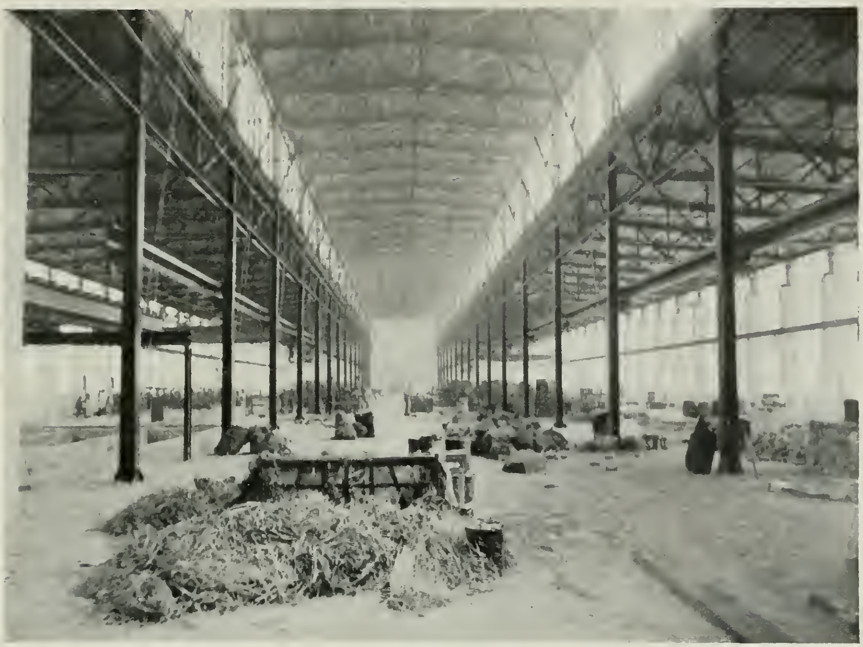
The factory has a total length of 541½ ft. and a width of 116 ft. 4 in., divided, as far as the works portion is concerned, into three bays. The central bay is 33 ft. wide with a clerestory roof 39 ft. 6 in. from the floor level. The side bays are 41 ft. wide with a height of about 25 ft. from floor to roof boards.

The whole of the floor is raised about 4 ft. 6 in. above the ground level in order to be on the level of the bottom of a railway wagon when on the adjacent sidings. This floor is carried upon 550 "Williams" patent steel and concrete piles, 25 ft. in length, firmly driven into the subsoil of gravel.

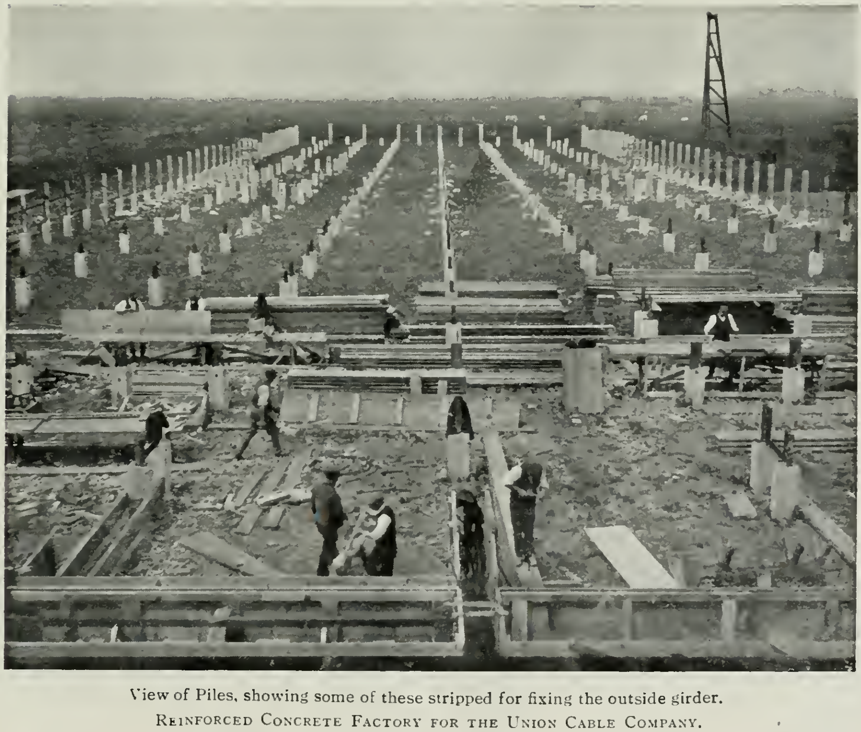


View of work in course of construction.

REINFORCED CONCRETE FACTORY FOR THE UNION CABLE COMPANY.



Interior View.



View of Piles, showing some of these stripped for fixing the outside girder.

Reinforced Concrete Factory for the Union Cable Company.

It will be seen from the illustration on page 778 that the piles support a reinforced concrete girder. The concrete has been broken away from the top of the pile allowing the central joist to become exposed. A fresh I-section joist has been fished on and stands up through and above the reinforced girder ready to be encased in concrete and faced on the outside with building blocks. It then becomes a column to support a roof principal.

The same procedure is adopted in the office block, and in the same illustration the columns will be seen supporting the various floors and prepared for taking the reinforced girders which carry the roof.

The roof is of reinforced concrete 6 in. thick and has sides which enable it to be used as a water tank containing some 60,000 gallons.

The walls of the office block are composed of hollow concrete blocks tied to the columns with hoop iron ties. They are supported at the bottom by the main girder and above each row of windows a ribbon girder of reinforced concrete incorporating the floor reinforcement is run all round the building. This ties the work together, forms window heads, and supports the block-work above it.

A reinforced concrete loading platform topped with scoriæ blocks runs the whole length of the building.

The roof of the main factory is composed of Ruberoid laid on 2½-in. tongued and grooved battens supported on steel principals which, in turn, are carried by the reinforced columns before mentioned on the outside and by two rows of steel stanchions on the inside. A good idea of the interior is shown in the illustration on page 779. In this the crane rails supported by reinforced concrete corbels are seen.

REINFORCED CONCRETE BRIDGE AT KING'S CROSS STATION.

The Metropolitan Railway have recently carried out important alterations and improvements in several of their stations, and it is worthy of note that reinforced concrete is being extensively used for these works.

The new bridge at King's Cross Station, uniting the Gray's Inn Road to the Pentonville Road, through the site of the old booking office, is the most recent application of this method of construction. The whole of the work was carried out under the superintendence of Mr. W. Willox, M.I.C.E., Engineer of the Metropolitan Railway Company. This bridge is a further proof of the confidence which engineers are now willing to give to reinforced concrete construction. The bridge, which is built on the Coignet system, is claimed to be the largest road bridge of this description in London. The various dimensions of the work are as follows:—

The bridge is composed of two spans of straight beams and is slightly on the skew, the main span is 52 ft. and the smaller span 32 ft., giving a total length, taking into account the short-end spans, of 130 ft.; the width of the bridge, including the footpath, is 60 ft.

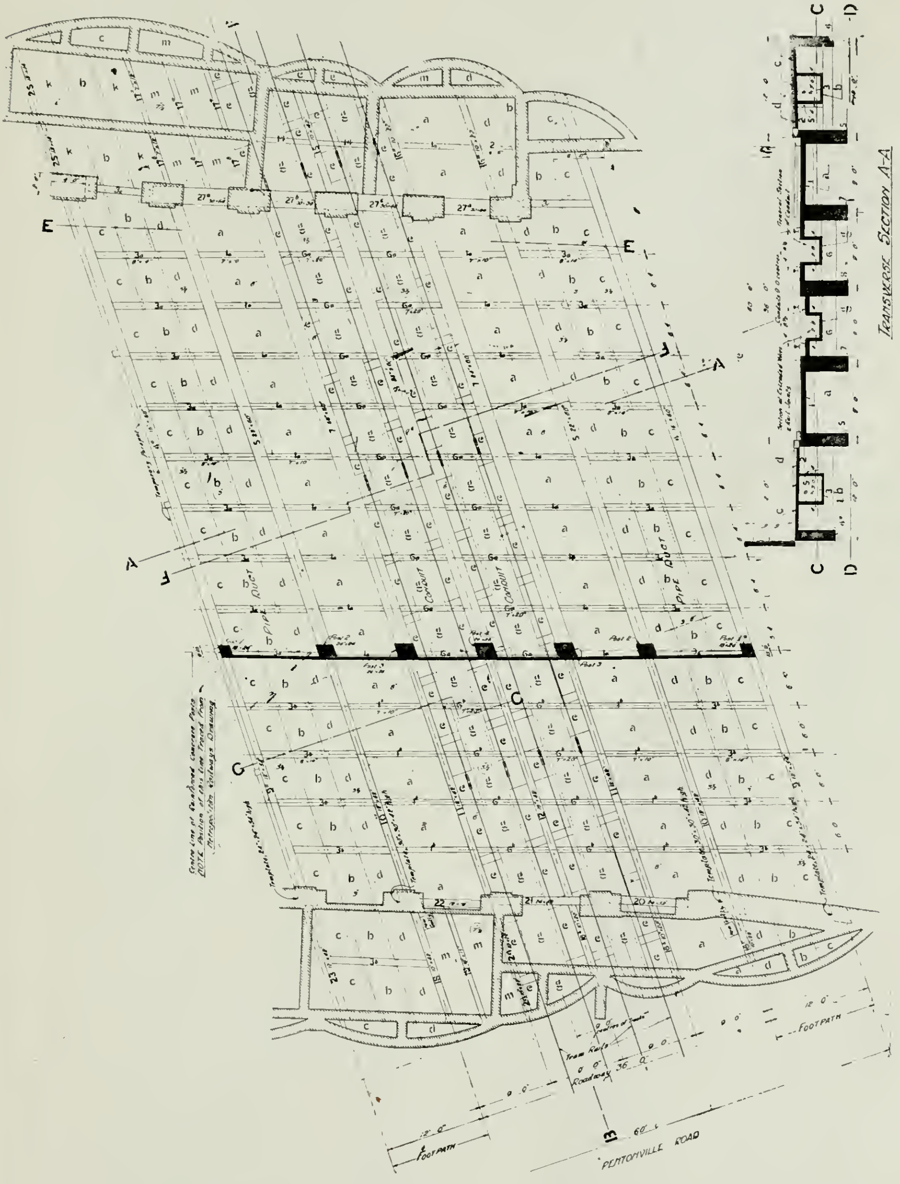
A remarkable feature of the construction was that it had to be erected without in any way interfering with the traffic of the four lines of railway below. In order to obviate the difficulty of having to support the wooden centering and moulds for the reinforced concrete over the railways, old trellis-work girders were used which had previously been utilised for the floor of the booking hall of the old station.

The bridge is calculated to carry two lines of double-deck electric tramways of the usual London County Council type. As shown on page 784, special accommodation has been provided for the cast-iron yokes and ducts supporting the rails and the transmission of the electric power. Special pipe ducts have also been provided for underneath the footpaths.

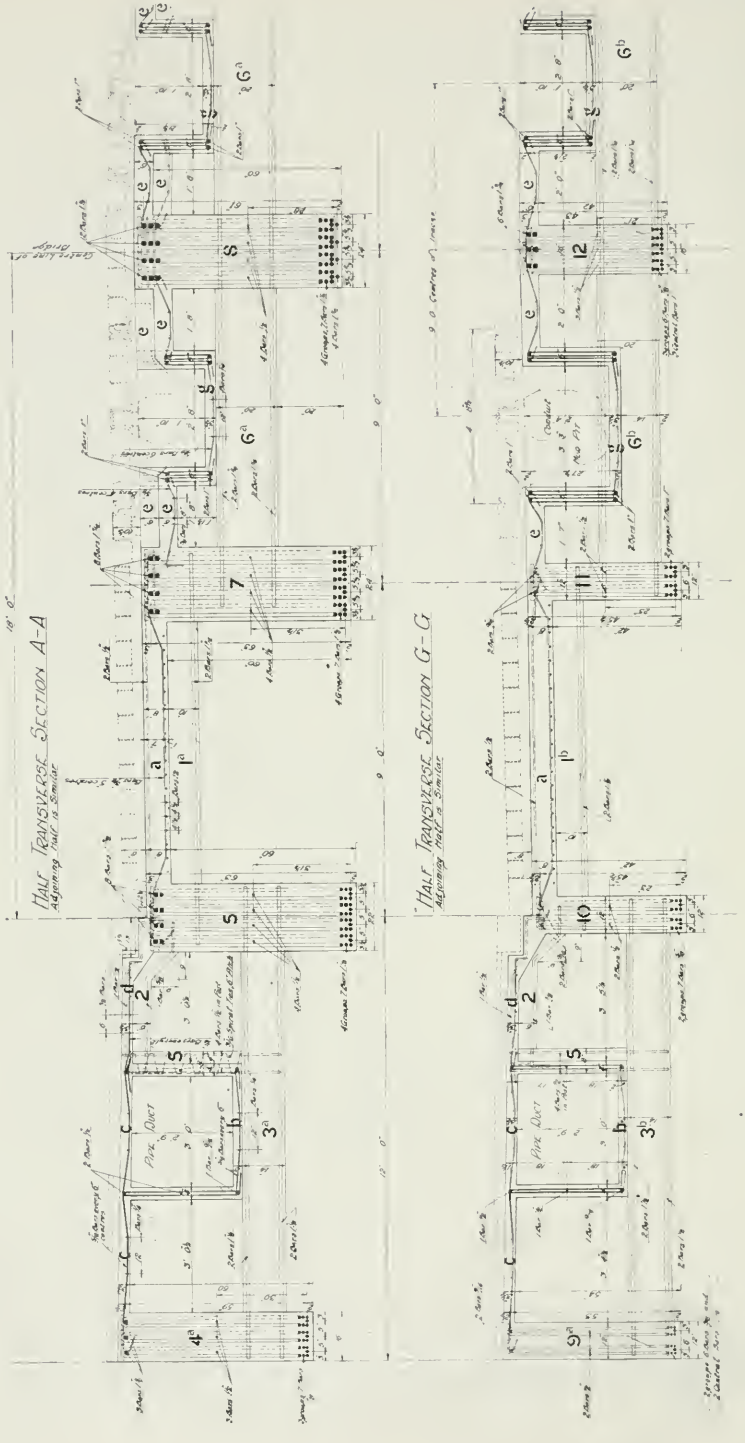
The roadway on either side of the tram-lines has been calculated for heavy traffic.

This bridge, which was opened to traffic on the 1st of July, was previously tested during the night whilst the traffic underneath was suspended. This test was carried out under the superintendence of Mr. O. C. G. Drury, engineer representing the Metropolitan Railway, and Messrs. G. C. Workman and M. Béhar, engineers representing Messrs. Edmond Coignet, Ltd. The contractors, Messrs. W. King and Son, were represented by the foreman, Mr. Selby. Special tramcars loaded with bags of sand were provided for the purpose by the London County Council, who also were represented by one of their officials. These loaded cars were gradually made to pass over the bridge at various speeds. No appreciable deflections, however, were recorded, and a noticeable feature was the entire absence of vibration, even when both trams were made to pass rapidly over the bridge.

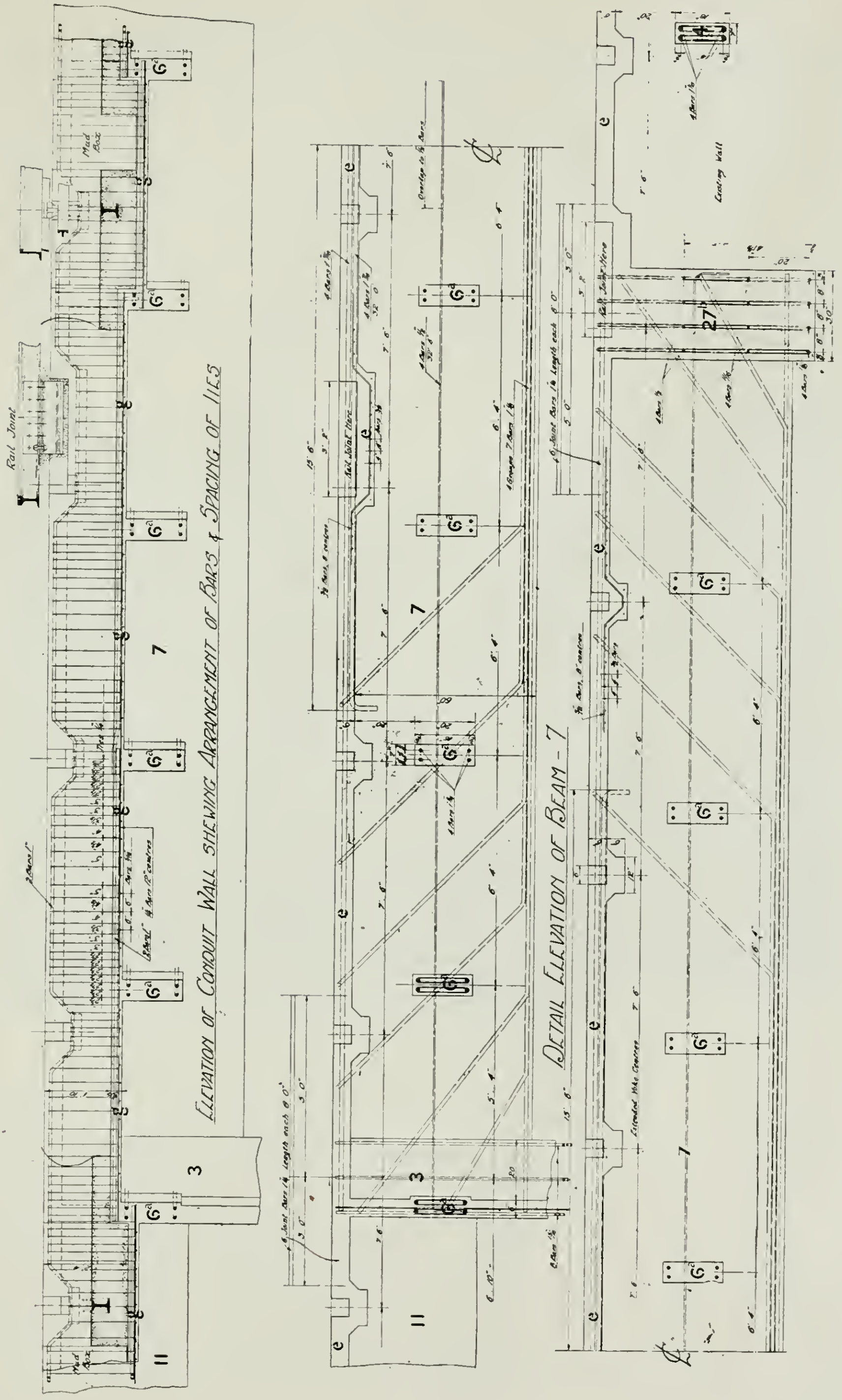
The reinforcement of the work is composed of round bars of mild steel arranged in accordance with the Coignet system. The main bars of the beams vary in diameter between about 1 in. and 1¼ in. The tension and compression reinforcing bars which are placed respectively at the bottom and at the top of each beams are connected together by means of stirrups, made also with round bars of small diameter. The bars in tension are arranged in groups of seven for each unit, the ends of six of the



Sectional Plan.
Reinforced Concrete Bridge at King's Cross Station.


HALF TRANSVERSE SECTION A-A
Adjoining Half is Similar
HALF TRANSVERSE SECTION G-G
Adjoining Half is Similar
Centre Line of Bridge
18' 0"
9' 0"
12' 0"
Pipe Duct
Conduit
4a
3a
2
5
1a
7
6a
S
9a
3b
10
1b
11
6b
12
a
b
c
d
e

Transverse Sections.
Reinforced Concrete Bridge at King's Cross Station.


Elevation of Conduit Wall shewing Arrangement of Bars & Spacing of Ties
Detail Elevation of Beam - 7
Rail Joint
Mud Box
Existing Wall
7
3
11
27
14

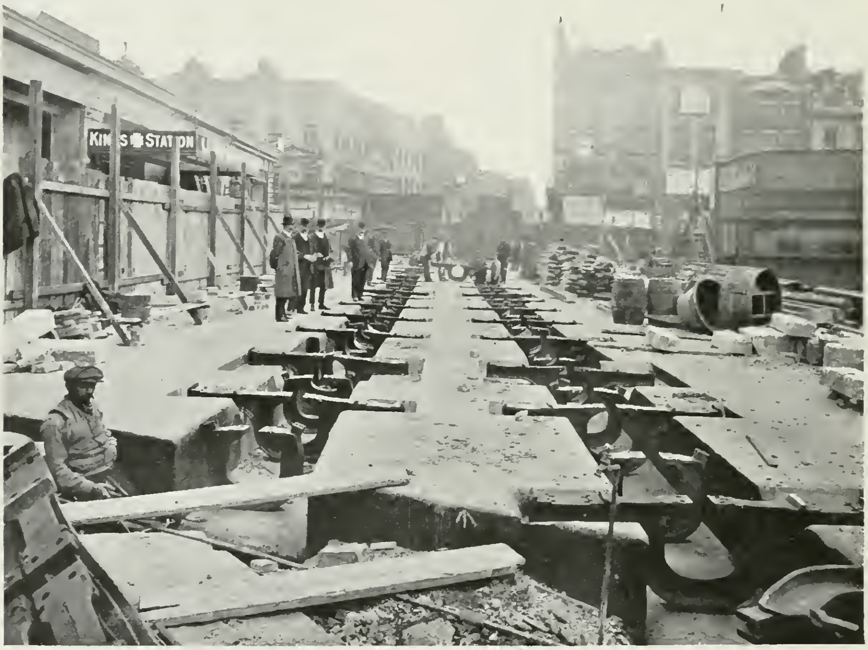
Elevation of Conduit Walls and Beams.
Reinforced Concrete Bridge at King's Cross Station.

bars of each group being gradually bent upwards and hooked to the upper bars of the beam in order to resist the shearing stresses and in order to form a mechanical connection between the top and bottom reinforcement. The slabs between the beams are reinforced by means of a mesh-work of round bars of diameters varying between $\frac{3}{8}$ in. and $\frac{5}{8}$ in.

The calculations of the work have been made in accordance with the rules of the Royal Institute of British Architects for reinforced concrete.

The design of the bridge was, we understand, obtained by Messrs. Edmond Coignet, Ltd., of 20, Victoria Street, S.W., in open competition with several other of the principal systems, and their scheme and calculations were very carefully gone into by the engineers of the Metropolitan Railway Company before deciding upon the adoption of the Coignet system.

It is obvious that very considerable precautions had to be taken by all those concerned in the construction of this bridge, more especially on account of the important position which it occupies, and the fact that it was impossible in any way to interrupt the traffic either underneath the bridge or at either side of the abutments during construction of the work. It is interesting to notice that, although there is a constant and considerable vibration practically on all sides of this bridge, this does not seem to have interfered with the concrete in any way. It is scarcely necessary to point out that this bridge in reinforced concrete will require no maintenance or painting as in the case of a steel bridge, and this is of some importance on account of the fact that, apart from the electric railways, there are two lines of ordinary railway traffic passing underneath the smaller span, and it is well known that the fumes of locomotives have the effect of deteriorating the flanges and web of steel girders.



View showing Conduits for Tramways.
REINFORCED CONCRETE BRIDGE AT KING'S CROSS STATION.

Another advantage of the use of reinforced concrete for this bridge is the lack of vibration as compared with a steel structure, and also the fact that instead of growing weaker with age the work will gradually increase considerably in strength.

CONCRETE BLOCK COTTAGES AT TALBOT'S INCH, KILKENNY.

In the frontispiece to this issue we show an illustration of six of the cottages at Talbot's Inch, Kilkenny, built for the Countess of Desart for the employees in the Kilkenny woodworkers, Kilkenny woollen mills, and tobacco plantation industries.

The cottages were built entirely of "Winget" blocks, the interiors all being Fibro cement sheeting and the roof of Fibro cement tiles. The cottages present a very pleasing appearance, as will be seen from the frontispiece, and have given complete satisfaction.

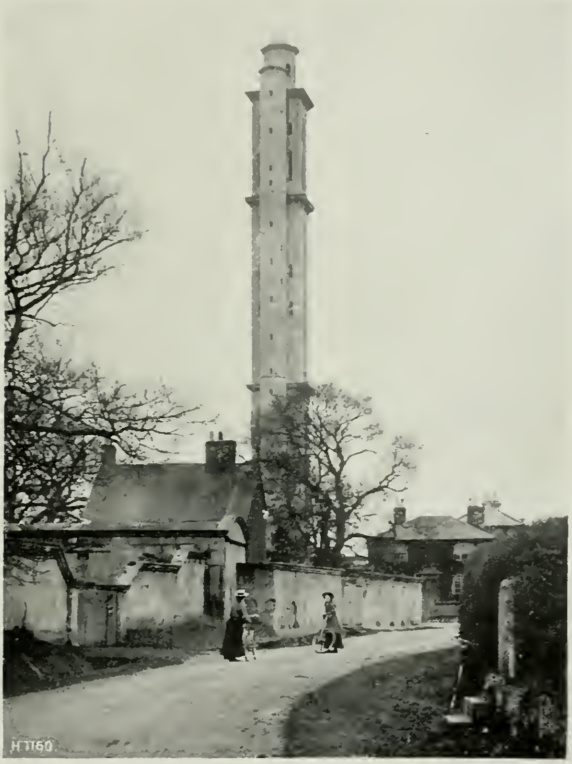
The whole of the cottages have been designed and carried out by Mr. F. W. Kiddie, Architect and Manager, Talbot's Inch.

THE HIGHEST SEPULCHRAL MONUMENT IN EUROPE.

There is situated at the little village of Hordle, some four miles from Lymington, in Hampshire, what is believed to be the highest sepulchral monument in Europe. This monument forms a landmark for miles round, and inasmuch as it is stated to be built of cement concrete the illustration of this curious erection which we are able to present will no doubt be of interest to readers of this journal.

The tower is stated to be about 300 ft. high, and to have been erected by the late Mr. A. T. T. Peterson some forty years ago to prove the durability of concrete as a building material.

Mr. Peterson died a few years ago, and left instructions in his will that his remains were to be deposited in this tower, where they now rest.



Peterson's Tower at Hordle, in Hampshire, built of Concrete.
The Highest Sepulchral Monument in Europe.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"Handbook of Reinforced Concrete Construction" (Handbuch für Eisenbetonbau). 2nd Edition. Vol. I. The History and Theory of Reinforced Concrete. Edited by Dr. F. von Emperger.

Berlin: Wilhelm Ernst & Sohn, 1912. Price 25 marks. Pp. xix.+655, with 975 figures and a plate.

The greater part of this standard work on reinforced concrete being now out of print, a second edition is in course of publication, of which the first volume is now before us. The work is thoroughly German in its conception and execution, and is planned on a scale which would probably be impossible in any other country. When complete, Dr. F. von Emperger's "Handbook" will occupy twelve volumes, or, with a supplementary volume dealing with the artistic treatment of reinforced concrete, thirteen. This first volume, admirably printed and produced, deals with the history of the development of reinforced concrete construction, and with the general theory, together with a review of the principal tests and experiments on which that theory is based.

It may be said at once that the treatment of the subject is encyclopædic, and the mass of the work is so great that it is not to be used or understood at all readily without a previous acquaintance with the subject, whilst some sections show an impartiality in recording the results of tests which is almost excessive, and the reader would be grateful for a more critical comparison of the results of different workers whose experiments are not conducted on a uniform plan. The value of the Handbook is, however, incontestable, and the sections of the volume may now be noticed in succession.

History.—The historical review is written by Prof. M. Foerster, of Dresden, and deals with the subject of Portland cement as well as with those of concrete and reinforced concrete. The author is fair in his examination of rival claims, and his narrative is both clear and interesting. More might have been made of the knowledge of concrete possessed by the Romans, whose use of the material was more extensive than might be supposed from the account given, whilst we now know that even reinforcement with metal was resorted to on occasion. In respect to cement, a definition of Portland cement is given which would exclude the so-called "iron-Portland cement." A highly interesting account of the development of reinforced concrete construction in each of the European countries and in the United States follows, in which full justice is done to the chief pioneers of the system. The methods of arriving at rules for computation are also briefly reviewed from a historical point of view, as a preliminary to the detailed treatment later on. Here we cannot but observe how small a share this country has had in the scientific development of this method of construction, in comparison with that of France, Germany, Austria, or Switzerland. There are, of course, many reasons for this, of which the almost complete divorce between architecture and engineering in England is one of the most obvious; but the fact is a rather melancholy one. The review of English work might have been somewhat more complete. The Concrete Institute is not mentioned, and the literature references only include the first edition of Marsh's well-known book, which appears in the bibliography as if it were published only in New York. Reference might also have been made at this stage to the publications of the Fire Prevention Committee, although the tests of this body will no doubt be discussed when the subject of security against fire comes to be treated.

Compressive Strength.—The compressive strength of plain and reinforced concrete is discussed in an illuminating manner by Prof. M. R. von Thullie, of Lemberg. In addition to the Continental tests, much use is made here of the excellent American tests of Talbot and others, whilst a few British tests are also included. Fuller particulars as to the exact nature of the concrete mixtures employed would be useful in certain of the cases quoted, but this important factor is duly considered in the author's summing-up. It is to be noted that the Austrian regulations provide for a permissible stress which varies with the materials used in the concrete.

Arches.—A very detailed account of tests on arches is given by Dr. Kleinlogel, of Darmstadt, from Wayss's experiments in 1886 to the most recent loading tests. The theory of the arch is treated in a separate chapter by Prof. J. Melan, of Prague. The highly mathematical character of this chapter may come as a surprise to those who have not yet realised the exceedingly complicated nature of the stresses in a hinged or unhinged arch under eccentric loading, and the formidable array of integrals found necessary in the solution of such problems is a vivid illustration of the need of thorough scientific training in this branch of construction. Reinforced concrete is not a material to be applied by rule of thumb, and it must be emphasised that a full mathematical investigation is necessary if such structures are to be erected with safety and without an excessive waste of material. The difficult subject of several continuous arches is well treated, and the chapter closes with a series of concrete examples, several arched bridges which have been actually constructed being worked out in detail.

Beams.—Tests and experiments with beams are treated by Hr. Otto Graf, of Zuffenhausen. An interesting conclusion, drawn from the results of a number of workers is that whilst reinforced beams which have been allowed to harden in air show an initial tension in the concrete and compression in the steel, the reverse is the case if the beams harden under moist sand, the steel being then brought into tension and the concrete into compression. Much space is given to the question of the resistance of the reinforcement to slipping, the influence of the quality of the concrete on resistance being very strongly marked, whilst the figures quoted point to a great advantage in using steel which has not been deprived of its natural black surface, and of bending up the ends of the main longitudinal reinforcing rods. This is one of the sections where a final review of the data collected would have added greatly to the value of the chapter. In the author's opinion, the Prussian regulations lead to an over-estimate of the tensile stresses in the steel, whilst he is in agreement with other writers in stating that the elastic limit of the steel determines the resistance of a beam, provided that slipping does not take place.

The theory of reinforced beams is separately treated by Dr. P. Völker and Hr. E. Richter, who give a clear and readily comprehensible account of the theoretical basis of the computation of beams, especially as regards the validity of the assumptions which are necessarily made. It is considered that a satisfactory formula for taking account of the tensile stresses in the concrete in a reinforced beam has not yet been obtained, owing to the different moduli of elasticity of concrete for compression and tension. The use of the value 15 for the ratio of the elastic moduli of the two materials is justified, although that ratio is actually variable and depends on the nature of the concrete mixture employed, and on the degree of loading. The section dealing with the effect of variations of temperature on the stress in a beam omits to take account of the low conductivity of concrete for heat as compared with steel, and a consideration of this factor would somewhat reduce the amount of the secondary stresses due to temperature. The notations used in the formulæ, it should be noted, are not in accordance with those approved by the International Association for Testing Materials. A general agreement on this important point is very desirable. Heavy type is used for the more important formulæ, and both analytical and graphical methods are employed and explained, a large folding plate being added for use in graphical computations, besides smaller diagrams. All this section appears to be excellent. An elaborate and useful index, prepared on a systematic plan by Prof. E. Brugsch, of Hanover, concludes the volume.

It is not possible to make a comparison of such a work as this with any existing text-book, since it stands alone in its scale and comprehensiveness, and it is likely to remain for long the standard work of reference on the subject. It affords striking evidence of the thought, labour, and skill that have been devoted to the investigation of reinforced concrete on the Continent. Some useful British work has undoubtedly been overlooked, but when every allowance is made the fact remains that this country, in which constructional work in steel has reached so high a state of development, has not by any means contributed its due share to the advancement of a method of construction the advantages of which are now generally admitted. It is still possible that this defect may be remedied, and a perusal of this volume by those responsible for the training of engineers in this country should do much to bring about a realisation of the importance of a scientific study of the principles involved.

"Preliminary Studies in Bridge Design." By Reginald Ryves, *A.M.I.C.E. (U.S.A.), etc.*

London: The St. Bride's Press, Ltd., 24 Bride Lane, Fleet Street, E.C. 57 pp.+viii. 2/- net.

Contents.—The Five Primary Types of Bridges—The Problem as a Whole—Three Types of Streams—The Choice of a Site—The Fixed Site—The Stream with a Defined Flood Channel—The Bridge as a Check on Floods—The Number and Size of Arches—Wandering Streams—The Bridge at a Bend—The Pooling Method—Fords—The Road Dam—The Inverted Syphon—The "Overall" Span—Impact, Maximum Loads, Loading as Regards the Centre Line—Some General Comparisons, The Arch, Steel and Timber Bridges, Ferro-Concrete Bridges.

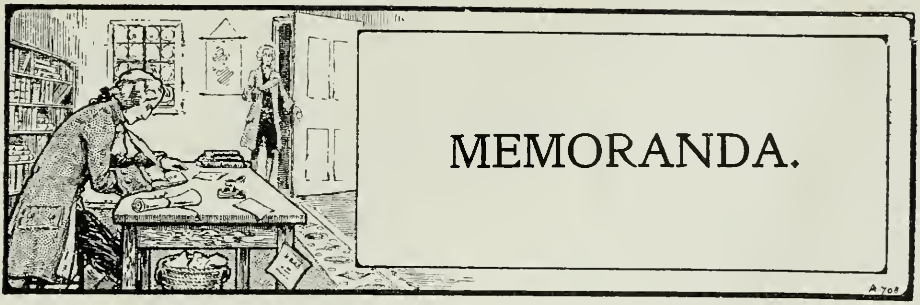
This is a small volume containing a reprint of a series of articles which appeared in *The Surveyor and Municipal and County Engineer* early in 1912, and it is an excellent little book written in an interesting manner. The author has dealt with the preliminary study of bridge design in a very thorough manner, and suggests to the reader the correct method of commencing the work by pointing out the many items that have to be considered before even the type of bridge is decided upon. The student should find the volume of value, and it will give him a good groundwork upon which he may build up a more advanced knowledge of the subject. It is a great thing to start with a thorough knowledge of the elementary principles, and we can then profit most surely by our subsequent experience. The author in dealing with ferro-concrete bridges says: "On the most pessimistic view—or almost that—ferro-concrete bridges may be expected to outlast steel bridges of the same or greater cost. On a more hopeful view they may last so long that they may prove satisfactory substitutes for solid masonry bridges." We must say that we hold the hopeful view and have no hesitation in saying that ferro concrete is certainly a satisfactory substitute for masonry and will last even longer. Again, there are possibilities with reinforced concrete that could never be attained with masonry alone, and there is no doubt that for bridges of practically all kinds, the former material is superior to all others. Bridge design offers excellent opportunities for producing work which is not only constructionally good, but also architecturally beautiful, and examples of some of the schemes that have been carried out in reinforced concrete—particularly where the open spandril type has been adopted—can be seen which are without parallel in any other material.

"Reinforced Concrete Construction." Vol. I. Fundamental Principles. By George A. Hool, *S.B.*

New York: McGraw-Hill Book Company, 239 Wes 39th Street. 254 pp. + x.

Contents.—Concrete — Steel — Concrete and Steel in Combination—Rectangular Beams—Slabs, Cross-beams and Girders—Columns—Slab, Beam, and Column Tables—Slab, Beam, and Column Diagrams — Bending and Direct Stress—Tables.

This volume deals very fully with the fundamental principles of reinforced concrete and is written and illustrated in such a manner that it should be understandable by those possessing a fair knowledge of mechanics. We regret, however, to find that no table of notation is given at the end of the volume, and this must be considered a disadvantage in view of the fact that it is intended to appeal to students; and as the notation used does not tally with that generally adopted in this country, it will be confusing to the reader who might be studying this volume in conjunction with other books on the subject, and who will be at a loss to find the value of an unfamiliar symbol without looking through a great number of pages in the hope of finding the notation containing the particular symbol in question American authors would be well advised if they considered the difficulty of the English student in following a formula which contains practically, to him, unknown symbols, and endeavours should be made to simplify this trouble as much as possible. In an elementary book of this kind we should also like to see the derivation of the formulæ put forward in a more simple manner in order that the construction of the formula itself may be properly understood. There are many good points in the volume, and we consider the diagrams are excellent and might with advantage be studied by some of the authors in this country. The notes on shearing stress and diagonal tension are good, and this is a portion of the subject which is far too often treated in an indifferent manner, and the notes on the calculations for deflection are interesting. It is a very complete work, dealing as it does with all the aspects of slab, beam, and column design, and, in spite of our criticisms, has much to commend it to those interested in the subject in this country.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

British Standard Specification for Structural Steel for Bridges, etc., and General Building Construction.—A new edition of the "British Standard Specification for Structural Steel for Bridges and General Building Construction" (Report No. 15. Revised August, 1912) has recently been issued. The new issue is of importance in view of the fact that the London County Council (General Powers) Act, 1909, requires that all rolled steel used in the construction of skeleton frame-work for buildings shall comply with the requirements of this specification.

Since its first issue, in June, 1906, the Committee from time to time had brought before them points which had arisen in connection with its use, and the present issue embodies the necessary amendments in regard to these. Some of the more important modifications to the Specification are as follows:—

1. The Classification of the material dealt with into two categories, A. Steel and B. Steel.
2. The increase of the allowable Phosphorus Content in B. Steel from ·07 to ·08 per cent.
3. The increase of the upper limit of tensile strength for plates, sections and bars (other than rivet bars), from 32 to 33 tons per sq. in.
4. The reduction of the lower limit of tensile strength for rivet bars from 26 to 25 tons per sq. in.
5. The insertion of Clauses dealing with (*a*) Tests by an independent expert; (*b*) rejection after delivery, and (*c*) arbitration in cases where this is not otherwise provided for.

Following the procedure adopted in the revision of the British Standard Specification for Portland Cement, the text of the Specification has been settled in consultation with the Committee by Mr. A. A. Hudson, K.C. (standing counsel to the Committee), with a view of ensuring that, so far as practicable, it has the meaning intended by them, and the Specification has been remodelled in accordance with his advice.

The price of the Specification is 5s. 2d., post free, and it may be obtained from any bookseller or direct from the offices of the Committee, 28, Victoria Street, Westminster, S.W., and is published for the Committee by Messrs. Crosby, Lockwood and Son, 7, Stationers' Hall Court, Ludgate Hill, E.C., and 5, Broadway, Westminster, S.W.

The Concrete Institute's Visit to Works in Course of Construction.—By the courtesy of Mr. A. Sykes, F.R.I.B.A., some of the members of the Concrete Institute recently paid a visit to the warehouse now in course of construction for Mr. J. Sainsbury, situate at the corner of Bennett and Stamford Streets, close to Blackfriars Bridge, S.E.

This warehouse, which measures approximately 165 ft. long by 54 ft. wide, comprises six floors and roof. With the exception of the elevations to Bennett Street and Stamford Street the whole skeleton is constructed in reinforced concrete, lintels being provided in the outside walls to carry the brickwork, which has a thickness of 2 ft. 3 in. at the bottom. The interior columns had to be kept down to as small a size as possible, and to carry the six floors heavily loaded a large percentage of steel was necessary, one of the lower columns requiring nearly 40 sq. in. In the ground floor three loading docks, 24 ft. by 20 ft. and 4 ft. 5 in. below ground floor level, have been provided, each designed to carry a ten-ton axle load.

Each floor consists of a $4\frac{1}{2}$-in. slab carried on secondary beams 5 ft. to 6 ft. apart. This $4\frac{1}{2}$-in. floor is composed of $3\frac{1}{2}$ in. of 6 to 1 concrete and 1 in. of granite finish laid simultaneously with the concrete. The secondary beams are 14 in. deep below slab and 7 in. wide, and have spans varying from 11 ft. to 15 ft. 5 in. The secondaries are carried on main beams 18 in. below slab, and 10 in. and 8 in. in width; the span of the main beams varies from 12 ft. to 22 ft. 6 in.

Another interesting feature in this work is the retaining wall and vaults below the road in Bennett Street and Stamford Street. To carry the heavy road traffic in Bennett Street a superload of 10 cwt. per sq. ft. was allowed on the roof of the vaults, and a corresponding lateral pressure on the retaining wall, and the lateral thrust on the building had to be carefully provided for.

The foundations are composed partly of independent footings to columns and partly of strip footings of various lengths and widths. Some trouble was experienced with the ground, and the foundations had to be carried lower down than at first intended, and in one case piles had to be driven to a depth of 12 ft. below foundation level.

The reinforcement throughout is Indented steel bars.

Protective and Decorative Treatment of Concrete and Cement Surfaces.—Concrete and cement surfaces should be divided under two heads—viz., weathering and wearing. The former includes such surfaces as walls, etc., and the latter floors. Concrete, in theory, is watertight, and of uniform texture, but in practice, although all precautions may be observed, these results are not always obtained. In the construction of monolithic buildings, and even with cement rendering, it is not possible to mix more than a small proportion of the concrete at one time. It is therefore quite possible that one batch may vary to a certain extent from another, both in proportions of aggregate and cement and in quantity of mixing water. These circumstances, taken into consideration with the placing and tamping, which is done by hand, often result in a somewhat uneven surface. Excessive tamping will force the finer particles of cement towards the surface of the form, thus resulting in a laitence face. Insufficient tamping, however, will often produce a very rough surface, in all probability showing the aggregate to a large extent. Further, laitence face is very liable to scale, owing to the difference in the expansion of this crust and the inner portions of the concrete. Surfaces such as this are admittedly most unsightly.

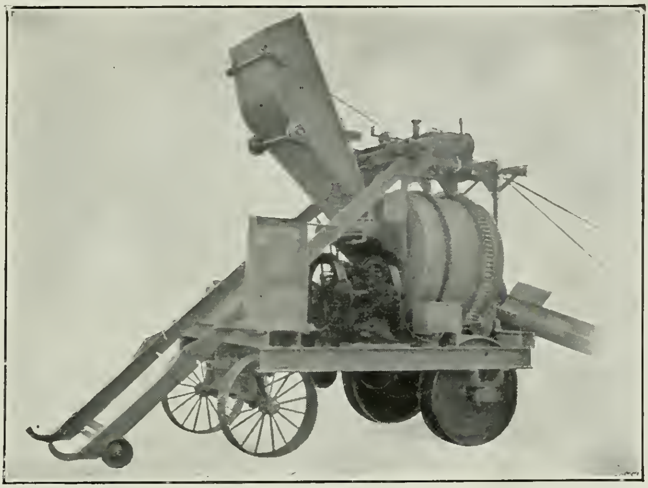
Porosity is also caused by the same circumstances, and is undoubtedly a very serious feature of concrete construction. These superficial voids of the surface are acted upon by atmospheric gases, moisture, etc., and tend to become filled with a crystallisation of the free salts in the concrete. This results in swelling, thus causing disintegration of the surface.

In order, therefore, to overcome the disadvantages of a plain monolithic concrete surface it seems necessary that a treatment should be given which is both protective and which will improve the appearance—that is, waterproof and decorate.

When mention is made of treating a new concrete or cement surface the question immediately arises as to the difficulty which has always been experienced in applying anything in the form of a paint to such surfaces. As is well known, protective coatings made with linseed oil as a medium cannot be applied within a period of at least eighteen months without the danger of saponification. It is quite evident, therefore, that this purpose must be met from a neutral substance which is totally unaffected by acids or alkalis. It must be at the same time elastic, to withstand weather conditions, and also with a natural stone-like finish.

Under the heading of a wearing surface the most important is undoubtedly that of concrete floors. They have to withstand a great deal of friction, and are peculiarly liable to dusting and sanding up. This may be due to the floor having been laid under extreme temperature conditions, and to various other conditions, amongst which may be mentioned the use of dryers, anti-freezing compounds, use of artificial heat for drying purposes. It may also be caused by too much trowelling; this causes the water to come to the surface, bringing with it the fine particles of cement, and producing a glassy looking surface, which will sooner or later scale off. Concrete floors, moreover, are not oil and greaseproof. Oil, grease, or acid, etc., will sometimes affect the cement. Therefore in factories, warehouses, etc., where delicate machinery is used or foodstuffs, leather, etc., stored, such conditions should be guarded against. It therefore appears that a protective coating is required that will actually penetrate the surface, and reinforce the grain bond of the concrete—namely, cement—at the same time producing a grease, oil, and acid-proof surface. Such a material would undoubtedly be of very great utility, and would supply a long-felt want of the architect, engineer, and factory owner alike.

A New Design of Street Mixer.—The Ransome Concrete Machinery Co., Dunellen, N.J., have just put on the market two new types of street mixers, the No. 60 and No. 61. There are many points about these which are different from anything which has been offered to date by mixer manufacturers, and the design has been made by the Ransome Company after a long study of the street paving contractor's problems. The No. 60 machine is a steam-driven outfit regularly furnished with 10-ft. distributing chute capable of handling 14 cubic ft. of loose material, and will mix 40 to 60 batches per hour, depending upon the speed with which the material can be fed to the pivot hopper. The engine is 8 h.p. and the boiler 9 h.p. The machine is arranged to drive in both directions along the work.



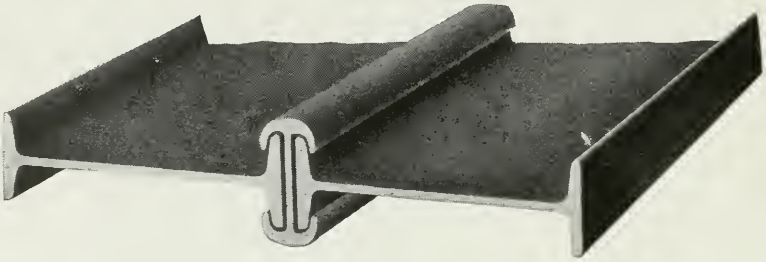
A New Design of Street Mixer.

The various levers controlling the operations of hoisting, discharging, and traction are all brought to one point, and the entire operation can be controlled by the engineer. The chute will swing through an arc of approximately 180°, while the discharge point is 3 ft. 7 in. above street level. The outfit is mounted on 24- and 28-in. wheels with 10-in. tyres. The net weight of the machine complete with power is approximately 11,000 lb.

The No. 61 Ransome street mixer will handle up to 20 cubic ft. of loose material, and will mix from 40 to 60 batches per hour. It is regularly equipped with a two-cylinder opposed gasolene engine and two 20-ft. distributing chutes. It is arranged with a clutch, by which the drum can be thrown out and remain idle while the machine is travelling along the road, and it is also equipped with a traction device adapted for moving in either direction. A notable feature of this outfit is the fact that the over-all height has been kept within 11 ft. 2 in. This has been done by eliminating the charging hopper and emptying direct from skip into mixer.

The driving wheels are 44 in. in diameter and the front wheels 40 in., the tyres being 10 in. and 8 in. respectively. By using these large wheels an 18-in. clearance above the ground has been made possible, thus avoiding any jacking up of the machine to clear manholes which are liable to project 12 in. or more above the sub-grade of an unfinished street. When desired flanges may be bolted to the tyres, thus making the outfit suitable for operation under the machine's own power on trolley tracks, etc. If the outfit is to be drawn behind a trolley car at high speed standard flanged wheels may be readily substituted for the ones regularly furnished, as the wheels are easily taken off, inasmuch as the driving axle is mounted in split boxes and the front axle can be swung around and the wheels slipped off.

UNIVERSAL JOIST
STEEL
SHEET PILING



THOUSANDS OF TONS IN USE. USE A PILING THAT HAS BEEN THOROUGHLY TRIED AND TESTED. DO NOT BE LED AWAY BY FREAK AND INEFFICIENT SECTIONS OF LIGHT WEIGHT & WEAK INTERLOCK

* * *

THE BRITISH STEEL PILING Cº
Dock House, Billiter Street
London, E.C.

Telephone
1414 Avenue

Telegrams
"Gramercy, London"

All of the operating levers have been brought adjacent to the operating platform at one side of the machine. The steering wheel is in front. The line shaft is driven by chain, and the power is transmitted through clutches to the windlass for operating the hoist, the drum, and the traction wheels. Of course, when desired an electric motor can be substituted for the gasoline engine, and when operated in railway service the motor could be run from overhead trolley wire. The general dimensions are as follows :

Weight	15,000 lb.
H.P.	16
Height over all, skip raised	14 ft.
Height over all, exclusive of skip	11 ft. 2 in.
Extreme width	8 ft. 6½ in.
Length, exclusive of skip and chute	17 ft. 1 in.
Length with skip lowered	20 ft. 10 in.

Concrete Sea Defence Work at Hastings.—It is pleasing to note that the quantity of beach on the foreshore at Hastings is increasing at several points where more is most required. There is one spot, however, where, although there is now more beach than there was a short time ago, still more of this natural protection against the force of the waves is required. It is at Caroline Place. This spot is one of the sights of the borough when the sea is rough, being a favourite resort at such times for those who care to face the elements. The Town Council are now wisely taking steps to secure adequate protection for this exposed portion of the Front, and it is proposed to place a new concrete groyne between two of these works already in existence at a cost of £450. If the work is carried out it should greatly strengthen the sea defence works at a point where the sea comes in with great force.

Reinforced Concrete Bathing Place at Devonport.—The Devonport Sanitary Committee have accepted the tender of Mr. Lester, of Plymouth, at £879 for building a retaining wall of reinforced concrete for the open-air bath at Mount Wise, and for covering the floor of the bath with concrete. In connection with the scheme it is proposed to erect twenty-four dressing boxes at an estimated cost of £320.

Experiments have been made, says *The Engineer*, to ascertain directly the lateral pressure of liquid concrete. A large number of experiments were made on different types of concrete structure. In heavy walls, large piers and other members of fair size the lateral pressure exerted was found to be fairly uniform, and practically constant for equal heads; but in reinforced concrete columns of small dimensions, thin walls and other light concrete work, the effect of friction between the more or less rough timber forms and the concrete, together with the arching action, was found to reduce the pressure considerably. The results lead to the conclusion that for average conditions the lateral pressure of concrete is equivalent to that of a fluid weighing 85 lb. per cubic ft. For concrete in which little water is used in mixing the pressures are rather less, having an equivalent fluid value as low as 70 lb. per cubic ft. in very dry mixtures.

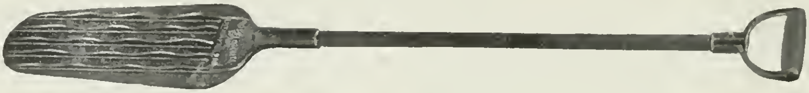
Reinforced Concrete Bridge at Wergin's Bridge, Herefordshire.—The Herefordshire County Council has accepted the tender of the Trussed Concrete Steel Co., Ltd., of £1,550, plus £150 for the approaches, for building a reinforced concrete bridge at Wergin's Bridge, this Company giving a two years' guarantee as to the design and efficiency of their construction.

Building Construction in China.—According to the current issue of the *Board of Trade Journal* the United States Consul-General at Hong Kong, in a recent report to his Government, states that methods of building construction in Hong Kong and South China ports have undergone a great change during the last year or two. Formerly most buildings were made of brick or stone, and the use of steel in construction was exceptional, but very few big buildings have been constructed recently in which structural steel was not used, and the use of reinforced concrete is becoming more and more general. European metal is preferred for concrete reinforcement of heavier types, but American materials are chiefly used for the reinforcement of floor, ceilings, and non-bearing walls, also for bridges, galleries, verandahs, etc. Plans for construction work of all kinds in both Government and private enterprises under the new *régime* at Canton and in other ports call for an increased use of these materials. There are no reliable data as to the present volume of this demand, but it is considerable, and will, according to the report, undoubtedly increase greatly in the near future.

TRADE NOTICES, ETC.

The "Ross" Concrete Spade.—A short time ago we had the pleasure of bringing before the notice of our readers the "Ross" Concrete Spade, with offset, which was introduced into this country by the (U.K.) Winget Concrete Machine Co., Ltd., of Newcastle-on-Tyne.

A new design of spade has now been brought into the market by the same company. The new pattern spade is made without offset, is specially designed for producing a perfect face up to the shuttering in reinforced concrete walls and vertical work generally, and has proved to be a great money and labour saver.



THE "ROSS" CONCRETE SPADE.

It can be supplied either with long or short handle at the same price at purchaser's option. An illustration of spade with short handle is shown above.

CHANGES OF ADDRESS.

Messrs. Stuart's Granolithic Co., formerly of 4, Fenchurch Street, E.C., have removed to larger and more convenient premises at 45, Bedford Row, London, W.C.

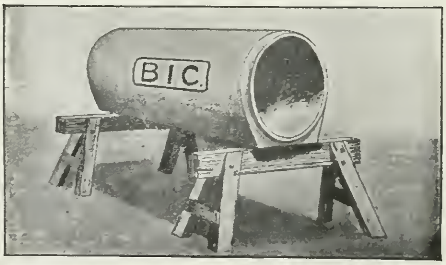
Messrs. W. & G. Foyle.—Owing to the increase in the extent of their business, this well-known firm of second-hand booksellers have found it necessary to take larger premises at 121-123, Charing Cross Road, W.C. Any of our readers desirous of studying the literature of reinforced concrete and engineering subjects generally, will find a large stock of up-to-date volumes at these premises.

"BIC" BRITISH "BIC"
IMPROVED CONSTRUCTION CO., LD.
TELEPHONE: 4067 VICTORIA. TELEGRAMS: "BICONCRETE, LONDON."
47 VICTORIA STREET, WESTMINSTER.

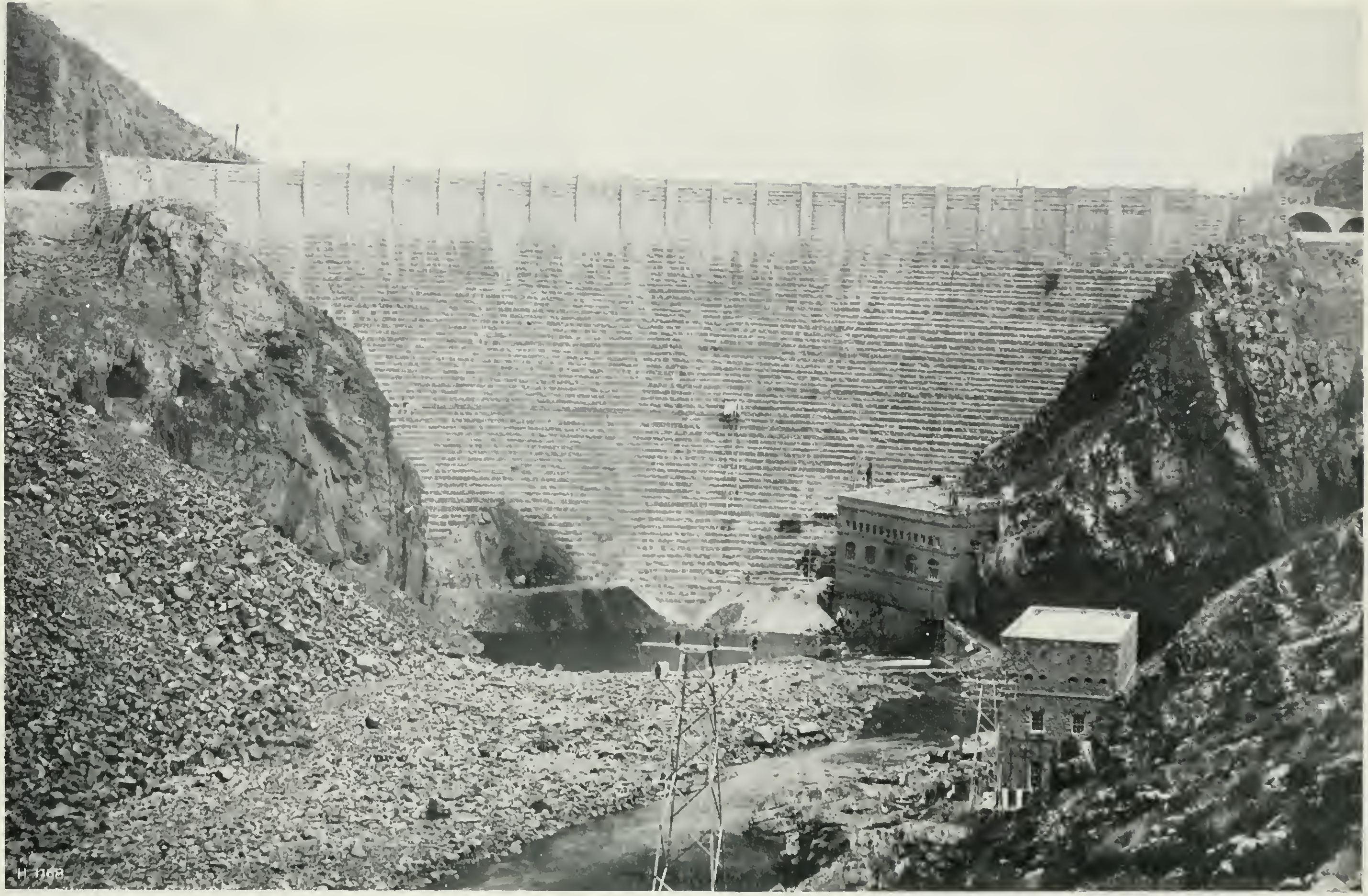
Flat-based Concrete Pipes
Three feet long to Nine feet long.

Breeze Partitions
Reinforced Sleepers
Telegraph Poles

Porous Pipes
Concrete Beams
Hollow Floors



SPECIALITY:—INVENTORS' DESIGNS CARRIED OUT TO ORDER.



(For description see page 821.)

The Roosevelt Dam.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume VII. No. 11. LONDON, NOVEMBER, 1912.

EDITORIAL NOTES.

THE INTERNATIONAL CONGRESS ON THE TESTING OF MATERIALS.

IN further reference to the International Congress held in New York in October, we have now received a copy of the resolutions adopted at this great gathering, and below we present such of these resolutions as have bearing on the subjects in which this journal is interested. It will be observed that three of the resolutions refer to Portland Cement, one to the Weathering of Building Stones, one to Plastic Mortars, and two to Reinforced Concrete; further, there is one regarding Fire-Resisting Construction, and, finally, there are two as to some International Technical Expressions.

Regarding the resolutions appertaining to Portland Cement, it will be observed that they are somewhat of an academical character and invite further investigation. The same may be said regarding the one dealing with the Weathering of Stones and Plastic Mortars.

The resolution relating to Reinforced Concrete and Building Accidents, however, is creative and beneficial, and we trust that the Concrete Institute will do its share in co-operating with the other nations on the lines indicated. It is certainly advisable, as the resolution says, " that a uniform system of reporting on building accidents should be adopted in the different countries," in the same way as there is considerable uniformity in reporting on boiler accidents, and there is also no doubt that if authentic records of accidents were duly prepared and published, they would act as a useful preventive against future accidents.

We welcome the tenor of the resolution relating to Fire-Resisting Building Construction, for it would certainly be useful if the International Association for Testing Materials were to accord the subject of fire-resistance of concrete and reinforced concrete as used for building far greater attention than heretofore. It is, of course, pleasant to observe that the British Standard Tests of the British Fire Prevention Committee are to serve as a basis for the testing conditions, and that a special sub-committee has been appointed by the Association for Testing Materials to make further investigations into the problems that call for inquiry. The resolutions say that " the data required should refer specially to the loss of strength during fire; the loss of strength after a fire (both upon rapid cooling and natural cooling) "; further, " that the value of different aggregates and the proportion of Portland cement to the aggregates " should be observed from the fire point of view, as also " the value of the different forms of reinforcement."

Quite apart from any question of aggregate, it is obvious that the different forms of reinforcement vary materially in their fire resistance, and the somewhat unwise arguments put forward in certain quarters that the mere fact that floor A or floor B has withstood the standard test is sufficient proof that floor C or D will do likewise are, of course, entirely fallacious. Although one system of design may be more or less identical with another system and possess an almost equivalent amount of fire resistance, material differences in design of the same floor or the same beam mean a very great difference in fire resistance, and frequently the difference is so great as not merely to mean a difference in the extent of resistance, but just the difference between success and absolute failure.

In closing our remarks as to the excellent work done by the International Association for Testing Materials, we wish to take the opportunity of congratulating Professor N. Belelubsky, of St. Petersburg, upon his election as President of the Association in the place of Professor M. H. Howe (Past President), and we also note with great interest that the next Congress is to take place in St. Petersburg in 1915.

THE CONGRESS RESOLUTIONS

CONSTANCY OF VOLUME OF PORTLAND CEMENT.

That the task of further examining all methods of accelerated tests for the constancy of volume of Portland cement be referred to a Committee and that it shall be an instruction to this Committee to invite experiments by various testing laboratories, and also to have some experiments made by an official testing authority in each country concerned.

FINE PARTICLES IN PORTLAND CEMENT.

That Committee No. 30 for determining a method for ascertaining the fine flour in cement be requested to endeavour to ascertain the approximate size in which the individual particles of cement have to be reduced in order that the particles may hydrate completely at short periods.

SO_3 IN PORTLAND CEMENT.

That the Council be requested to appoint a Committee to report at the next Congress regarding the effect of SO_3 in Portland cement.

WEATHERING VALUE OF BUILDING STONES.

The Congress decides that the different countries be invited to have the method of Hirschwald examined in testing laboratories and that the results be reported to the next Congress.

REINFORCED CONCRETE.

That Committee 41 be instructed to report at the next Congress on Methods of Determining the Quality of Concrete and of Reinforced Concrete.

PLASTIC MORTARS.

That Committee 42 be invited to continue its work in order to obtain a uniform method of testing plastic mortars and to apply this method to the study of the comparison of various standard sands.

REINFORCED CONCRETE BUILDING ACCIDENTS.

That with a view to the prevention of accidents and facilitating the acquisition of the requisite knowledge of the properties of materials, it is advisable that a uniform system of reporting on building accidents should be adopted in the different countries, in the same manner as is already done in the case of accidents to steam boilers. The Congress expressed a desire that Committee 41 shall endeavour to organise such an international system of reporting and to present to the next Congress a report on building accidents, collected country by country, together with the deductions made therefrom as regards the prevention of accidents.

FIRE-RESISTING BUILDING CONSTRUCTION.

I. That it is advisable in the future investigations of the strength of concrete and of reinforced concrete as used in buildings to obtain reliable data of the effect of fire at high and at moderate temperatures upon these materials.

II. That the data required should refer especially to:

1. (*a*) Loss of strength during fire.

(*b*) Loss of strength after a fire. (Both upon rapid cooling and natural cooling.)

2. (*a*) The value of different aggregates and the proportion of Portland cement to the aggregates.

(*b*) The value of different forms of reinforcement.

III. That tests conducted with the view of obtaining these data in different countries be arranged as far as possible on a uniform basis, the universal standards for fire tests of the British Fire Prevention Committee adopted at an International Congress in 1903 to serve as far as practicable as a basis for the testing conditions, these conditions having also been largely applied in the tests of the Columbia University Testing plant at New York and elsewhere.

IV. That the Council of the International Association for Testing Materials create a special sub-committee (under Group B, Cement, Concrete, etc.) to deal with any problems arising out of the foregoing, to be known as the Sub-Committee on the Fire Resistance of Concrete and Reinforced Concrete, and that this Sub-Committee be requested to report at the next Congress.

SOME TECHNICAL EXPRESSIONS.

1. That the Congress recommends for the total or absolute force the terms:
In French: Force, effort (de traction, de compression, de cisaillement);
In German: Kraft (Zugkraft, Druckkraft, Schubkraft oder Scheerkraft);
In English: Force, load (tension, compression, shear, pull and thrust).

2. That the Congress recommends for the forces (efforts) per unit of area:
In French: Tension, pression, cisaillement (tension normale; pression normale, tension tangentielle);
In German: Zugspannung, Druckspannung, Schubspannung;
In English: Tensile stress, compression stress, shearing stress.
The terms: Frottement intérieur, innere Reibung, internal friction, shall be reserved for designating the tension, parallel to the plane in which gliding occurs, opposing this slipping.

THE CONCRETE INSTITUTE.

In our Memoranda we give particulars of the coming Session of the Concrete Institute. In the same chapter we refer to some educational lectures arranged by the Institute, and in another column we deal with the volume of *Transactions* recently published and giving an account of the Institute's work during the first six months of this year.

We have from time to time dealt at considerable length with the work of the Institute, and we wish it every possible success in the coming Session, when the work will be conducted under its new President, Mr. E. P. Wells, J.P. We trust that little or no time will be accorded during this Session to what we have generally termed *administrative* questions, but that the whole of the time and all efforts will be devoted to actual scientific and practical work, the scope of which is enormous, and upon the fringe of which alone the Institute has so far only touched.

It is to the Science Committee of the Institute that the professional world will principally look, and it is a matter of congratulation that the chairmanship of this important committee is in the able hands of Professor Henry Adams, M.Inst.C.E., who will be most effectively seconded by Mr. E. Fiander Etchells, F.Phys.Soc. (vice-chairman), whose many extensive contributions to the discussions of that body are well known.

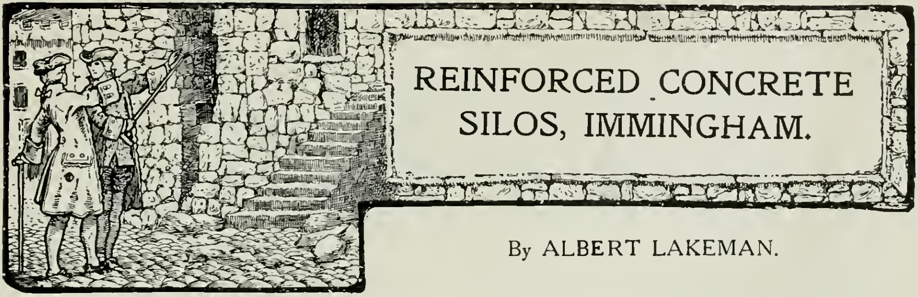
Apart, however, from the scientific aspect of the Institute's work, we would press for more work on the practical Parliamentary and legislative side, with due regard to the regulations for London, the provinces and the colonies.

The question of regulations for concrete and reinforced concrete has occupied the closest possible attention of public officers and the technical professions concerned, and there is no doubt that, whilst on the one hand greater facilities are required for the erection of reinforced concrete structures, such facilities should only go hand in hand with careful regulation and supervision from competent authorities. Here again, however, the Institute is most fortunate in having as its chairman of the Parliamentary Committee Mr. H. Percy Boulnois, M.Inst.C.E., whose former association with the Local Government Board gives him a wide cognisance of both the advantages and disadvantages of legislation, of *red tape*, and the difficulties that have to be contended with.

In concluding, however, we see no reason why both these committees should not add materially to the utility of the Concrete Institute during the impending Session, and it is to be hoped that they will both see their way to do active and energetic work in the interests of the subjects which come within their purview.

THE PORTLAND CEMENT INDUSTRY.

The annual general meetings of the shareholders of the leading cement manufacturing concerns in this country were held in October, and we are pleased to note that notwithstanding the reverses to which the trade has been subjected during the past year—mainly, if not entirely, owing to the unfortunate labour troubles which have been experienced—a promising year appears to be in prospect for the various companies. What concerns us, however, more than the financial aspect of the position, is the fact that it is the evident intention of the English cement trade to maintain the predominating position they hold in the world's markets by leaving no stone unturned to provide Portland cement of the highest quality at the lowest cost, and that whatever capital may be necessary to accomplish this end will be readily forthcoming. Beyond all question the British cement industry as a whole has kept well abreast of the times in the all-important matter of quality, and in some respects—notably in fineness of grinding—it is well ahead of the cement produced abroad. The tendency of modern times has been towards the concentration of the manufacture in fewer and, we believe, more capable hands, and in no small measure the improvements which we have from time to time noted have arisen from this fact. Portland cement of the highest quality is so essentially the very foundation of all constructional works that we welcome any event which may lead to an improvement either in its composition or in its method of manufacture, and in our judgment the recent developments which have occurred in the English trade clearly point to further advances in these respects.



REINFORCED CONCRETE SILOS, IMMINGHAM.

By ALBERT LAKEMAN.

We publish below some interesting particulars of some silos recently erected in reinforced concrete. We think the Great Central Railway Company are to be congratulated on their far-sightedness and progressive policy in employing reinforced concrete so extensively in their new port.—ED.

THIS building, which has been erected for The Humber Commercial Railway and Dock Co. at Immingham, is an important example of a complete reinforced concrete scheme, and it has been carried out on the Wells' system by Messrs. Stuart's Granolithic Co., Ltd. Concrete has been employed throughout, even for the large cornice, and the only portion of brickwork employed was that applied as a facing at the bottom of the building to form a deep plinth. The scheme can be divided into three portions—viz., (1) the receiving-house, (2) the silos, and (3) the granary—each of which is more or less complete in itself and involving entirely different calculations and design. The receiving-house, which is connected to the silos at the back, is about 43 ft. by 27 ft. 6 in. and about 67 ft. high, and it contains three floors above the ground floor. This building is rather curious from a constructional point of view, inasmuch that although the walls, beams and columns are all of reinforced concrete, the floors are of timber 2 in. thick, carried on 11-in. by 3-in. joists, and the concrete beams carry the machinery and floor at points, whilst other concrete beams act merely as ties to the walls. These walls are, generally speaking, 6 in. thick for the two lower floors and 4 in. thick above this level, with horizontal and vertical reinforcement in both faces; with additional rods at the window openings, which occur on all floors. Rods are placed across the corners of openings in a diagonal manner to act as ties, and the wall reinforcement is turned around the bars at the openings and carried back in the concrete. Piers are formed at the corners of the building and at intermediate points, and these are 3 ft. wide and 12 in. thick, reinforced with four or eight vertical lines of reinforcement, the size being usually ¾ in. diameter, and ¼-in. diameter links throughout to each set of four rods. The interior columns are square and vary from 10 in. at the top floor to 16 in. on the ground floor, and these are reinforced with four rods, varying from ¾ in. to 1⅞ in. diameter, tied with ¼-in. links at 8-in. centres. The bases to these columns are generally 5 ft. square, having a minimum thickness of 6 in. splayed up to meet the column and increasing to 1 ft. 9 in. at this point, with ½-in. rods as reinforcement. The roof is formed with 4 in. of reinforced concrete carried on reinforced beams, which are graded to give the required falls, while ledges having a projection of 3 in. were formed as part of the walls to give a bearing for the floor joists.

The space occupied by the silos measures about 127 ft. by 74 ft. and there are eighty-two bins in all. These bins are 10 ft. square in the clear and

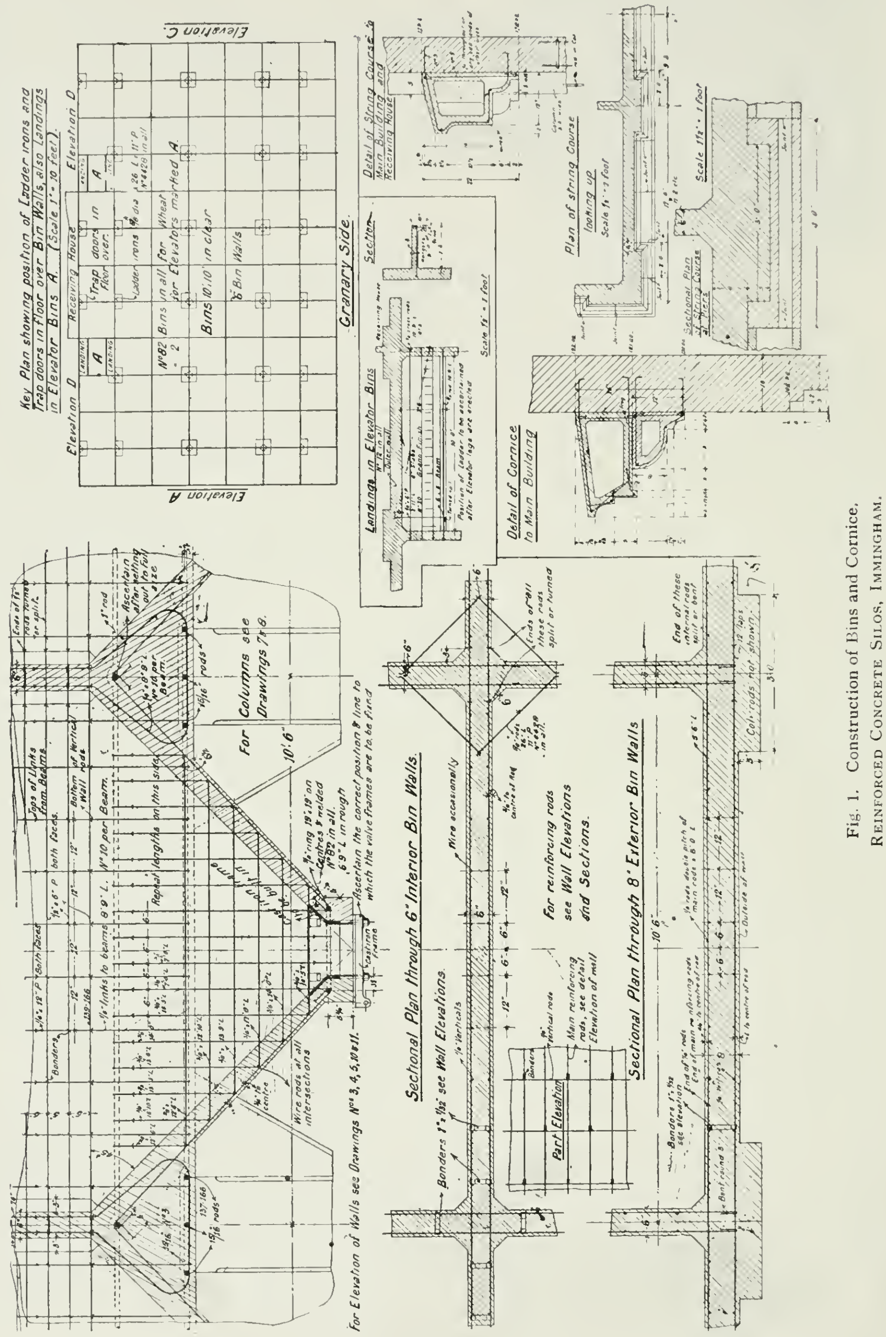

Key Plan showing position of Ladder irons and Trap doors in floor over Bin Walls, also Landings in Elevator Bins A. (Scale 1" = 10 feet).
Elevation A
Elevation C.
Elevation D
Receiving House
Trap doors in floor over.
Nº 82 Bins in all for Wheat
Bins 10'10" in clear
8" Bin Walls
Granary Side.
Landings in Elevator Bins
Section
Detail of String Course to Main Building and Receiving House
Plan of String Course looking up
Sectional Plan of String Course at Piers
Detail of Cornice to Main Building
For Columns see Drawings 7 & 8.
For Elevation of Walls see Drawings Nos 3, 4, 5, 10 & 11.
Sectional Plan through 6" Interior Bin Walls.
Sectional Plan through 8" Exterior Bin Walls
Part Elevation
For reinforcing rods see Wall Elevations and Sections.

Fig. 1. Construction of Bins and Cornice,
Reinforced Concrete Silos, Immingham.

have a height of about 50 ft., while they have a total capacity of over one thousand tons of grain. A weighing floor is constructed below these bins, giving

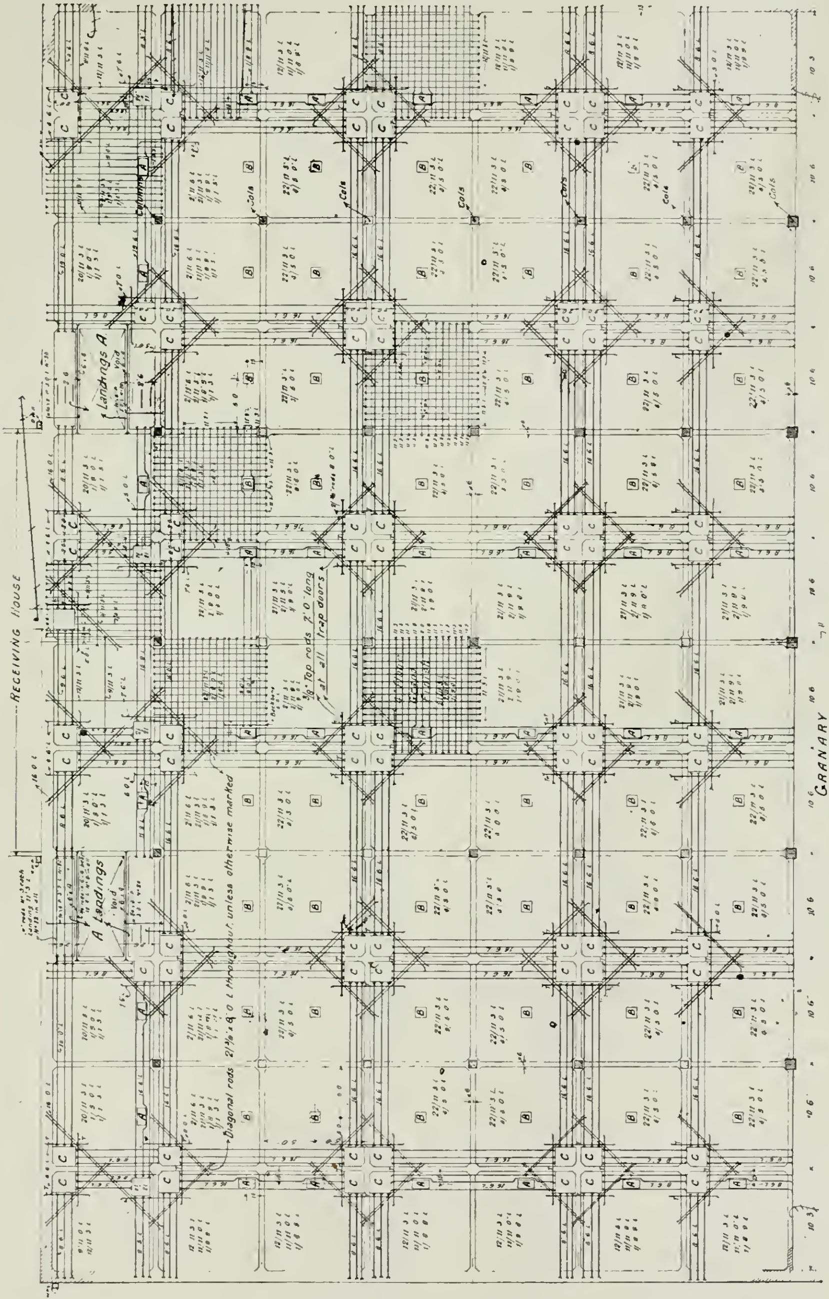

RECEIVING HOUSE
GRANARY
Landings A
A Landings
Void
Diagonal rods 2½" @ 0 L throughout unless otherwise marked
⅜" Top rods 7'0" long at all trap doors
Cols
C
A
B

Fig. 2. Floor over Bins, showing shoots and trapdoors.
Reinforced Concrete Silos, Immingham.

a total height of over 80 ft. to the parapet, exclusive of the towers, which are carried up a further 28 ft. above the parapet. The drawings illustrated in

Fig. 1 give some idea of the construction of these bins, where it will be seen that the division walls are 6 in. thick, reinforced with horizontal and vertical rods in both faces in such a manner that should one bin be full of grain and the adjoining bins empty the safe stresses would not be exceeded. All internal angles are finished with a 3-in. splay and ⅝-in. rods are provided at 11-in. intervals across the corners to form a climbing ladder in case it is necessary to descend into the bin. The bottoms of the bins are splayed to the outlet and have reinforced concrete varying from 4 in. to 5½ in. thick, with beams formed in the angles under the walls, practically of triangular form, with a width of 3 ft. at the bottom. The bars from the wall and the sloping bottoms are carried well into these beams, which are in turn connected to the columns.

Cast-iron frames are built into the bottom of the hoppers for the attachment of the discharge shutters, as shown in the detail. The external walls were formed with 8 in. of concrete in the lower 25 ft. and 6 in. above this level, with piers 3 ft. wide at 10 ft. centres, these latter projecting 5 in. on the outside of the building and having a flush face on the inside. The string course and cornice are shown in section in *Fig.* 1, and it will be seen that these were formed of reinforced concrete and are hollow in the interior. They were cast on a form of collapsible centre in convenient lengths and built in position as the walls were carried up. The detail is not quite correct as regards the attachment to the wall, there being a recess formed in the latter, which afforded a seating for the cornices, and rods were provided for tying back as indicated. As far as possible the joints were arranged to come over the centre of the piers, this giving an average length of 3 ft. 3 in. to each block. The floor over the bins was formed of 4 in. of concrete, reinforced with ⅜-in. rods in both directions, as illustrated in the drawing shown in *Fig.* 2, and finished with granolithic. The holes for filling the bins and the trap doors are also indicated, and it will be seen that additional rods were placed around these latter to effectually bind the whole together. The size and detail of the columns varied considerably, as shown in *Fig.* 3. Those attached to the walls as piers are generally 3 ft. wide and projecting 8 in. or 10 in. from the face of the wall, while the reinforcement consisted of eight lines of vertical reinforcement, well tied with links at about 8-in. pitch. Those under the bins vary from 21½ in. square to 22½ in. square and were reinforced with eight bars varying from 1½ in. to 1⅝ in. diameter, four of these being placed at the corners of the column, while the remainder were placed in the interior to form a square core with a 5½-in. side. The outer rods were tied with ⅜-in. links at 8-in. centres, while the inner rods did not require any binding on account of the large amount of concrete around them.

An interesting piece of construction is provided at the level of the weighing floor adjacent to the lines of rails that enter the building on both sides for transport purposes. A portion of the floor is cantilevered out for the full length of the building and a 4-in. wall is suspended from the ends of the cantilevers, which have a projection of 8 ft. 5 in. from the centre of the columns. These cantilevers have a depth of 18 in. and a thickness of 7 in. and they are reinforced in both top and bottom surfaces with four 1¼-in. rods connected by stirrups. The wall is formed with large openings in same, with piers or hangers between, and these latter are reinforced with four bars, while the panels under the windows have ½-in. rods in both directions as reinforcement. The columns from which the cantilevers project are slightly larger than the others, being 27 in. square, and these are reinforced with sixteen rods, twelve of which are spaced on the perimeter of the column and four within the core. It is rather unusual to find a wall carried in this manner, as it is literally "hung up" to the work above and has no actual seating or foundation. The plan of the granary is shown in *Fig.* 4, which is a drawing of the floors above the first floor.

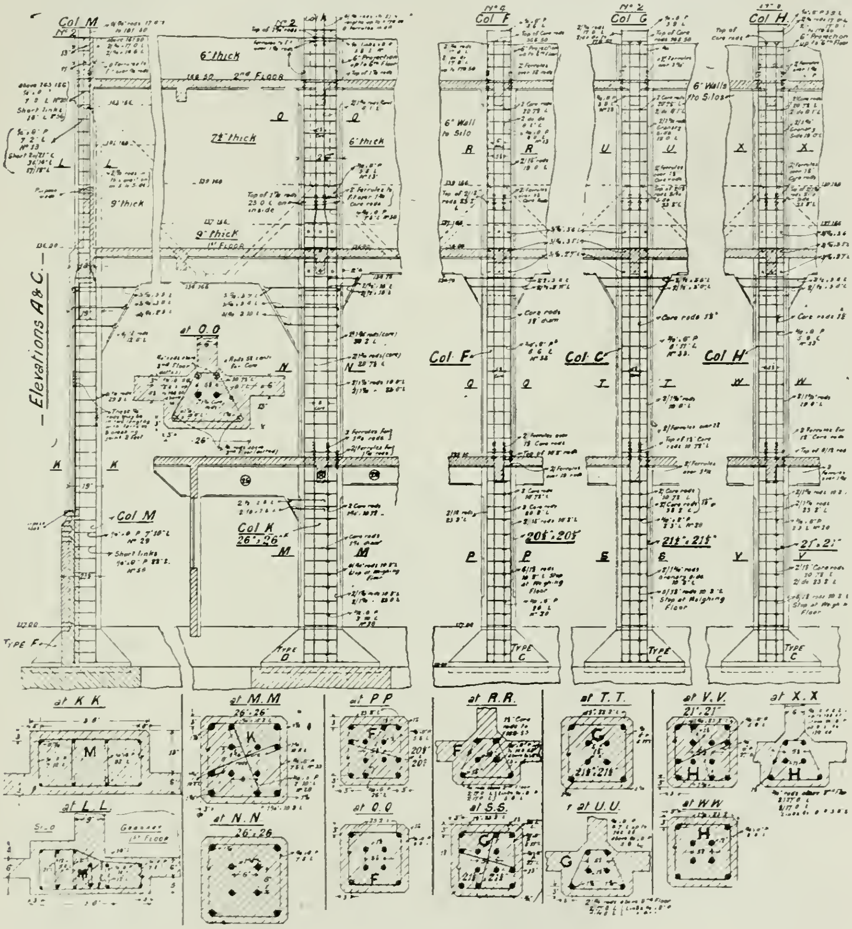


Fig. 3. Details of Columns.
Reinforced Concrete Silos, Immingham.

The building which adjoins the silos on the opposite side to the receiving-house measures about 63 ft. 6 in. by 127 ft., and there are six floors above the ground floor, giving a height equal to that of the silos. The floors were calculated to carry a load of 3 cwt. per ft. super, and these are divided up into bays 10 ft. 6 in. long and 7 ft. wide. These bays are carried by secondary and main beams, the former having a span of 10 ft. 6 in. and the latter a span

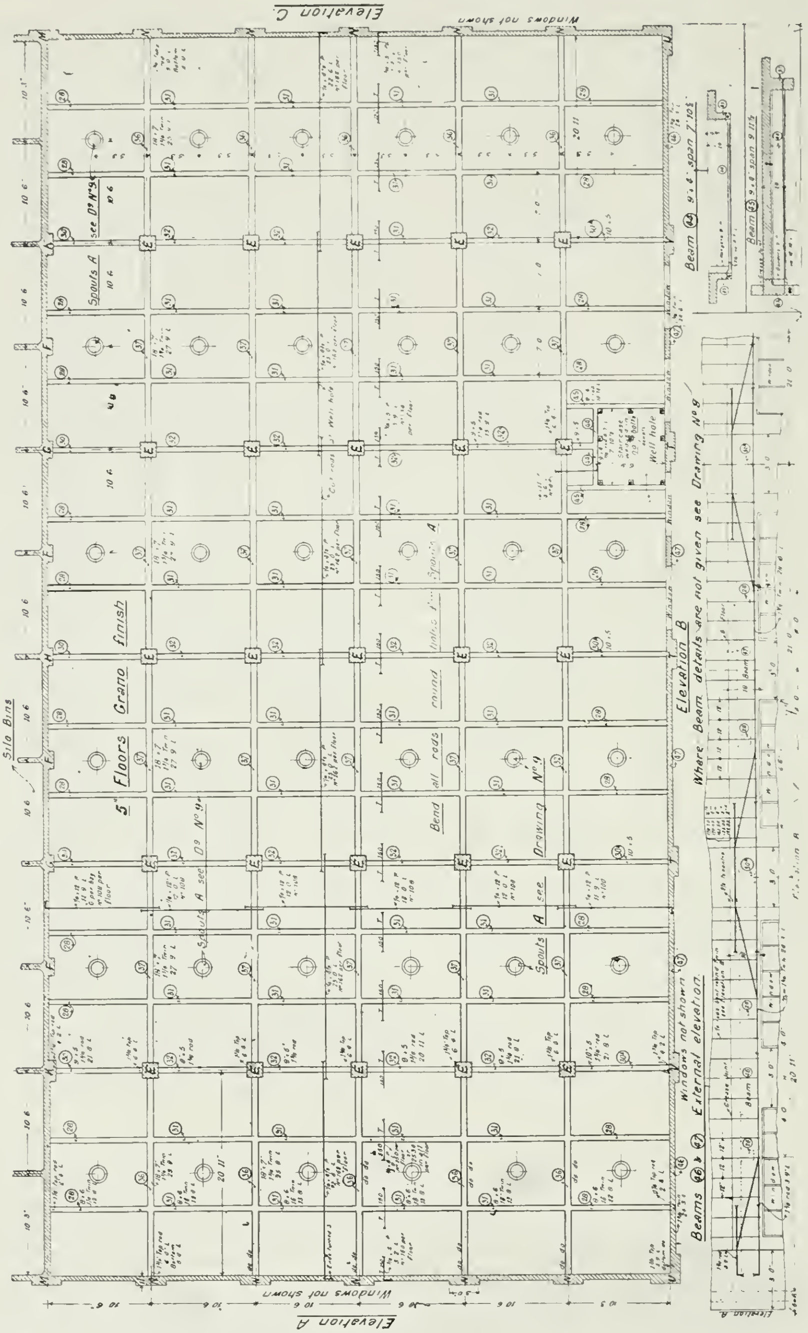


Fig. 4. Plan of Upper Floors to Granary.
Reinforced Concrete Silos, Immingham.

of about 20 ft. The slabs were made 5 in. in thickness and reinforced with

$\frac{1}{4}$-in. bars at $4\frac{1}{2}$-in. centres across the span of 7 ft., these bars being continuous

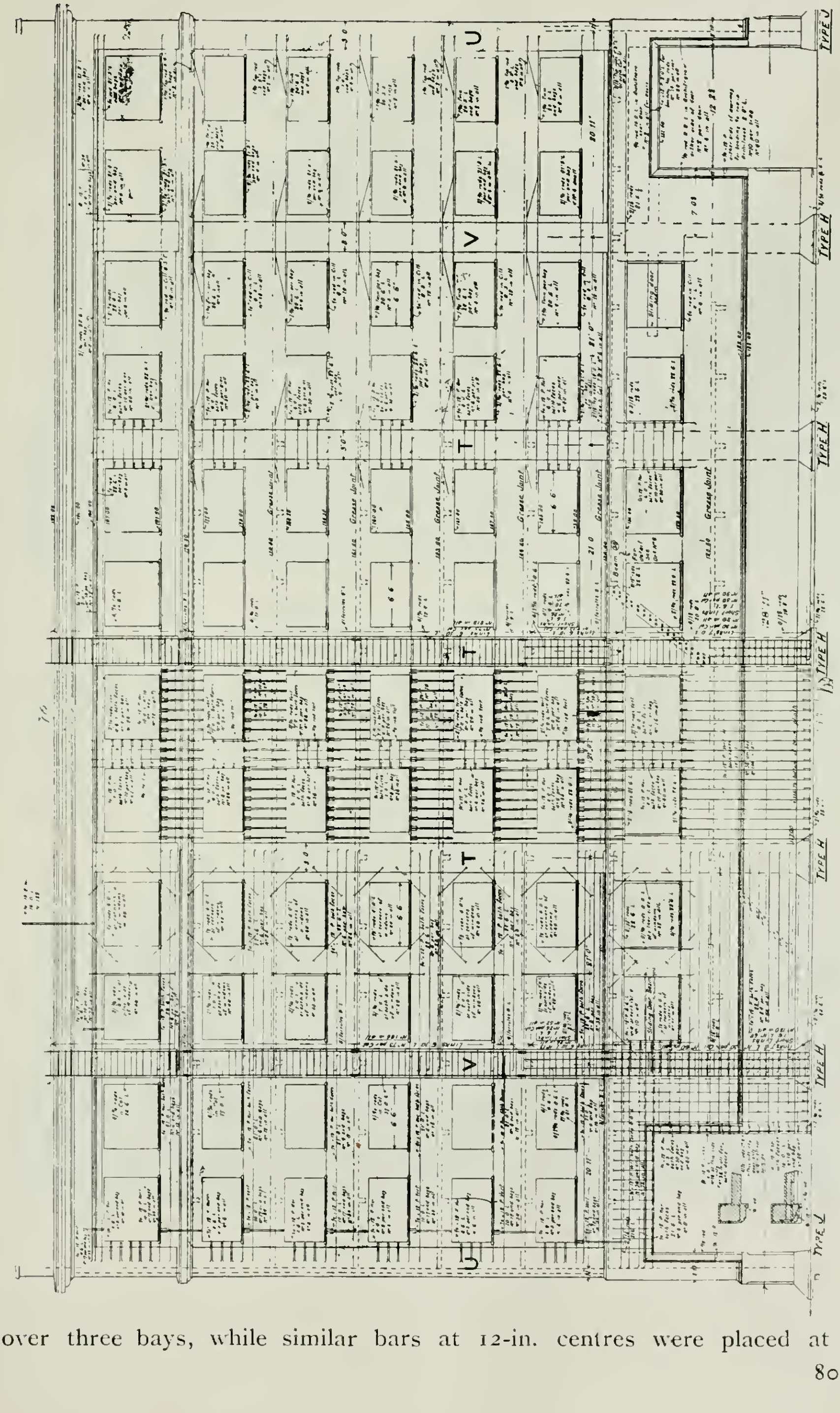


Fig 5. Construction of External Walls.
REINFORCED CONCRETE SILOS, IMMINGHAM.

over three bays, while similar bars at 12-in. centres were placed at right angles to these. The secondary beams were 8 in. deep and 6 in. wide, reinforced with one $1\frac{1}{8}$-in. twin rod, and hangers and bonders about 12 in apart,

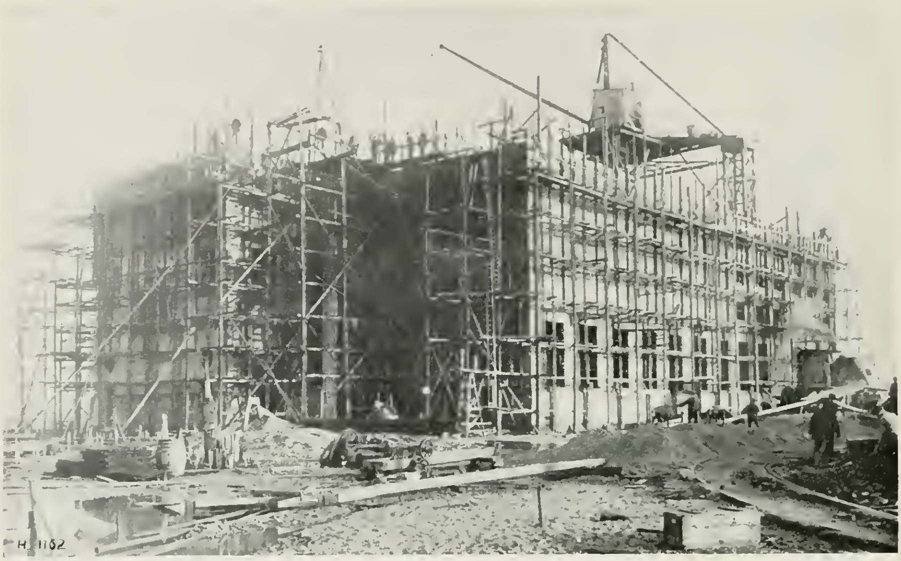


Fig. 6. View of Receiving End of Building.
Reinforced Concrete Silos, Immingham.

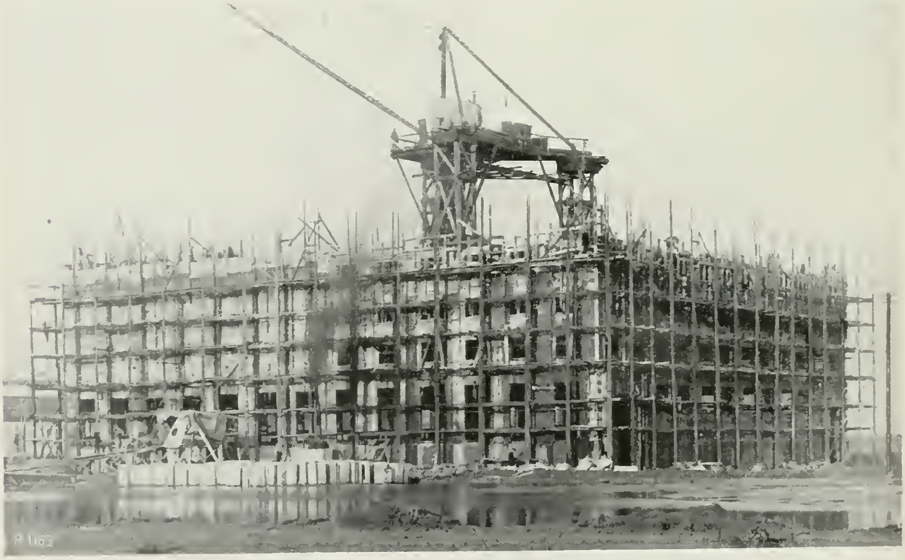


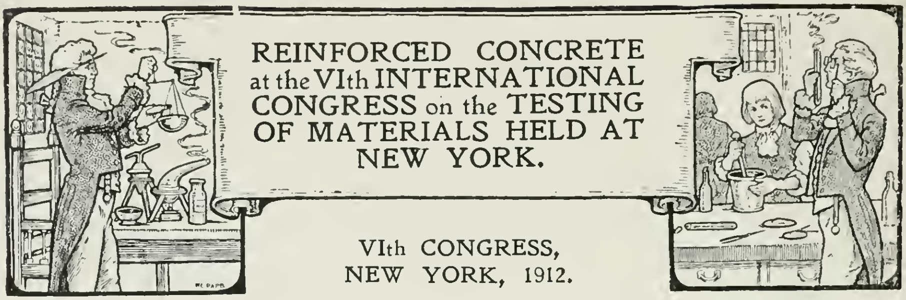
Fig. 7. General View of Work in Progress.
Reinforced Concrete Silos, Immingham.

while the main beams were 18 in. deep and 7 in. wide, reinforced with a $1\frac{3}{4}$-in. twin rod, with ample provision for shearing stress. The interior columns are

26 in. square, reinforced with fourteen vertical bars, ten of these having a diameter of 1⅝ in. and being placed near the outer surfaces, while the remaining four are placed within the core and have a diameter of 2 in., the whole being bound by ⅜-in. links at 8-in. centres. The external walls generally are 6 in. thick only in the panels, while the weights from the floor beams which come against the exterior are carried by projecting piers 3 ft. wide. The method of reinforcing the piers and panels is illustrated in *Fig.* 5, which shows the elevation of one complete wall, and the reinforcement around the windows, as previously mentioned, can also be seen. The towers which occur at the receiving-house end are carried up about 32 ft. above the cornice of the main buildings, and these are formed with 6-in. walls well reinforced and have a reinforced concrete string course and cornice constructed in a similar manner to that previously described.

The concrete throughout was machine mixed in the proportions of 9 cubic ft. of aggregate, 4½ cubic ft. of sand, and 224 lb. of cement, while all the floors were finished with granolithic composed of 5 parts of granite to 2 parts of cement.

The foundations are carried on wooden piles, which were driven before the commencement of the building contract proper, and these are in groups made proportionate to the loads to be carried, as many as eight being placed together in some instances. These are from 20 ft. to 30 ft. long, and the soil in which they are driven is practically a thick mud. The whole of the buildings have been carried out in an excellent manner by Messrs. Stuart's Granolithic Co., Ltd., from the designs of Mr. E. P. Wells, who acted as the consulting engineer to Messrs. Stuart's; whilst the engineer to the Great Central Railway Co., to whose instructions the work was carried out, was Mr. J. B. Ball. No other material could have given such successful results as those obtained by the use of reinforced concrete.



REINFORCED CONCRETE at the VIth INTERNATIONAL CONGRESS on the TESTING OF MATERIALS HELD AT NEW YORK.

VIth CONGRESS, NEW YORK, 1912.

In continuation of the Papers presented at this Congress, and printed in our September and October issues, we now give some further interesting contributions by Dr. von Emperger (Vienna), Mr. Albert Grittner (Budapest), and Mr. A. Martens (Berlin).—ED.

ACCIDENTS IN BUILDING WITH REINFORCED CONCRETE.

By DR. F. VON EMPERGER, Vienna.

Experiments in which accidents in the course of building are brought about by design are known to be capable of affording valuable information on the properties of materials; and thus an accident during building also forms a material test, carried to the point of rupture and furnishing particulars of a defective property of the material. It is naturally to the general interest that these tests should be confined to the laboratory, and we believe therefore that these experiments and the increased knowledge gained thereby on the building materials and their coaction form the means whereby accidents in practice may be prevented.

In order, however, to ascertain what are the points on which our knowledge is capable of improvement, and where experimental investigation should begin in order to attack the problem effectually, we must endeavour to trace the main origin of these building accidents and find in what their causes consist. The same dual task had to be faced in drawing up the report on building accidents, presented to the Brussels Congress in 1904. The inadequacy of private means for an exhaustive representation was expatiated upon in that report, and the desire was expressed that a clearer insight into the circumstances of the case might be afforded by the aid of unimpugnable official statistics.

This endeavour, the primary aim of which is the prevention of accidents, has, in the meantime, been able to record an important success, inasmuch as two countries have given effect to the aforesaid wish of the Congress by introducing a system of official reports, thereby creating an unimpeachable basis for further work of the kind. This was effected in a particularly thorough manner in Germany by an Ordinance issued by the Ministry of Public Works on September 18th, 1911, which, at the outset, was applicable only to Prussia (see "Beton und Eisen," 1911, p. 361).

This ordinance not only prescribed the procedure to be adopted in all kinds of building accidents (and therefore including also such accidents as, not being attended with any injury to life or limb, were not previously subjected to any jurisdiction), but also ensured competent judgment, by providing a list of experts, whereas the preexisting reports on building accidents were not only very imperfect, but had also been drawn up, as a rule, by persons more or less unqualified for the work.

A similar, though less detailed, ordinance was issued by the Austrian Ministry of Public Works at a somewhat earlier date—viz., on May 8th, 1911 (see "Beton und Eisen," 1911, p. 449). These two important ordinances fulfilled a principal wish expressed in the report of the Brussels Congress in 1906 in this connection, and at the same time created an influential example. At all events, they have restricted the system of private reports to the other countries exclusively.

Since the date of the last report only a few accidents, of little importance, have occurred to excite general attention. In view of the circumstance that these have all been reported in due course in the columns of "Beton und Eisen," the author considers it unnecessary to recapitulate them here, more especially since the published reports lay no claim to completeness and in many instances are derived from sources that are not always reliable.

One of the chief causes of such accidents has always resided in imperfect knowledge of the material at the time of removing the false work of the concrete, since, in view of the divergent influences to which the material is exposed in building operations, the quality of the material can only be imperfectly judged in the laboratory; or also because, in the absence of any connection between the laboratory and the building site, the material has actually escaped any checking. It has happened, for instance, that the false work has been taken down from concrete which has been spoiled by frost or checked in its development, although the regulations laid down for ordinary average conditions have been strictly complied with; and that this premature dismantling of the false work has led to extensive accidents. Moreover, it has also happened that some contractors have had accidents when working on proved lines through using materials to which they were not accustomed without having ascertained whether the same were equal to those with which they were acquainted.

Even the most careful precaution cannot, perhaps, always prevent accident; but builders do not always possess the warning consciousness of danger at the time when the latter is most imminent. For such contingencies the present writer has found a trustworthy and proved auxiliary* in the form of check or test beams, which enable one to ascertain, in a simple and reliable manner and on the spot, whether the concrete of a structure is quite ready for the false work to be taken down, or whether that operation should be delayed. The use of this auxiliary is strongly recommended when building operations are being carried on under unusual conditions, especially in autumn and winter, also in excessively hot weather and when unknown material is being used; and a warning must be uttered against the assumption, so frequently expressed, that an experienced concrete layer can judge concrete by touch. For instance, I have on several occasions on which specimens of sand and gravel have been rejected or declared from their appearance as bad by experienced men in the trade obtained fundamentally different results by means of the test beam. How much less is it possible to determine the influence of cold on the concrete from its external appearance only. For the purpose of affording a thorough check on concrete, I have applied and tested this method (for a description of which see "Beton und Eisen," 1903) during several building operations in Vienna in 1910; see more specially Kromus, "Die Betonkontrolle beim Neubau des k. k. Kriegsministeriums in Wien" (*Concrete and Constructional Engineering*, July, 1912), and also an exhaustive description by G. Neumann, "Eine Güteprobe für Beton, System Dr. F. von Emperger" (*Concrete and Constructional Engineering*, October, 1911).

Taking all these circumstances into consideration, the further continuance of the system of making private reports does not seem advisable; and it is recommended that the German and Austrian official representatives should be approached in order that a report from official sources may be available for the next Congress. The wish might also be expressed that the other States would follow the example and by means of these facts afford a comprehensive review of the measures to be adopted for the prevention of building accidents. The matter would become more complicated and important by the inclusion of the Latin countries, such as France, in which there is no stringent building-police supervision—at least of the kind customary in the Germanic countries. It would furnish an opportunity for comparing which of the methods gives the best results, or in which respect they have an improving influence on the contractor, if the statistics were compiled in an approximately uniform manner. The question at issue is whether a high condition of independence and the consequent feeling of responsibility are able to replace the present extensive but always insufficient supervision or not. The matter of uniformity of the reports demands an international pronouncement, which, however, is only possible on the occasion of our Congresses; and it would be desirable if this could be brought about by the time of the next Congress.

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* "II. Bericht der Tagung der höheren deutschen Baupolizeibeamten," Berlin 1911, published by W. Ernst & Sohn.

As one of the most important measures in connection with the widening of our knowledge on the matter in question, the fostering and extension of experiments in building-mechanics in general and in their simplest, and therefore so important form—namely, the test beam—are warmly recommended to all interested in the matter. It would also, in the first place, be desirable that reports on the use of the test beam in practice should be presented to the next Congress.

WATERPROOF CONCRETE.

By ALBERT GRITTNER, Budapest.

The application of fluates for the preparation of waterproof concretes has long been known. The operation requires much time and labour, for the desired object can only be realised by applying the fluates several times. Endeavours have hence been made to find other materials which would render concrete waterproof, when mixed to it, and which would be obtainable at lower cost and applicable with less trouble.

I have been investigating two such materials, namely "Biber" and "Aquabar." The experiments which I have been conducting have not led to any satisfactory results, although I applied the materials as prescribed by the manufacturers. These tests have, further, proved that Biber may be replaced by common coal tar oils, and that equally good, but equally less satisfactory, results are obtained with the latter.

The substitution for Aquabar, which is a soap solution, of a solution of an ordinary washing soap has not been successful. But when the common soap, which is a soda soap, was replaced by a potash soap I realised not only favourable, but indeed surprising results.

Before I pass to these experiments with potash soap I should like to refer to the results which I obtained in my fluate experiments.

For these experiments I used a Portland cement and a fine-grained artificial sand obtained from a Karst limestone. The sand consisted of 58 per cent. of grains less than 3 mm. in size and of 42 per cent. of grains more than 3 mm. in size. For the object of testing the fluates, hollow cubes were prepared of 7 cm. edge and 1 cm. wall thickness. In order to determine the height of capillary suctions, prisms were further made, 21 cm. in height and 5 by 5 cm. in cross section. Finally, discs were prepared, 9 cm. in diameter and 1 or 3 cm. in thickness, and were submitted to the water-pressure tests in a Böhme apparatus.

Portland cement tested in accordance with the standard Hungarian specifications yielded after 28 days the following strengths:

Tensile strength, 37·4 kg.; crushing strength, 444 kg.

The applied solution of magnesia fluate was of 20° Beaumé.

The specimens were made such that the density of the finished specimen was 2·31. The following are the results obtained :

Mixture ratio	Water, per cent. of weight of cement	Cement in kg. per m^3 of mortar	Waterproof in which condition	Consumption of fluate in kg. per m^2	Suction rise in cm. in the prisms after 24 hours	Water, in grs, penetrating under pressure of 2·1 atm. per cm^2 through disc thickness of		Water taken up by hollow cubes after 24 hours, in per cent. of weight of the cubes.	
						1 cm.	3 cm.	Per cent.	Note
1:3	50	458	Without fluate	—	6	0·78	0·41	19·6	Water height inside 3 cm.
			Fluate 2 times	0·4	5	0·31	0·24	9·2	Water height inside 1 cm.
			Fluate 4 times	0·7	3	0·12	0·04	1·2	Inner surface all moist
			Fluate 6 times	0·9	1·5	0·04	0·00	0·7	Moist in one spot

The prisms and hollow cubes stood in 4 cm. of water; the latter were covered with glass plates to prevent any evaporation of the infiltrated water.

The tests show that waterproofing can only be realised after applying several coatings of fluate, and that would render the process too expensive.

My potash-soap experiments were started on the instigation of Dr. C. Zielinsky, Professor at the Technical High School of Budapest, who had obtained good results when using potash soap on a concrete building.

The Portland cement used in these experiments showed after 28 days a

Tensile strength of 31·8 kg.; crushing strength, 440 kg.

The river sand used was fine-grained, 94 per cent. being less than 3 mm. in size. The density of the sand was 1·52. The gauge liquid consisted of a soap solution containing 8 kg. of potash soap in a 100 l. of water. It should be emphasised that the soap must be completely dissolved, which is best secured by taking care to use warm water and to filter subsequently through some loose fabric.

The density of the specimens was 2·22. After being kept damp for 28 days, the specimen was placed on a hot stove and completely dried. When the tests were finished, the specimens were once more dried at 120° C., and once more tested in order to ascertain whether the capacity for keeping the water out had not altered. The results were the same in the first and in the second instances. The dimensions of the specimens were the same as those used for the fluate tests. The following are the results :

Mixture ratio	Liquid in per cent. of cement	Cement in kg. per m^3 of mortar	Crushing strength after 28 days	Suction rise in prism in cm.		Water column of 85 cm. sinks in the disc, 1 cm. in thickness, to cm.		A water pressure of 2·1 atm. forces through disc of 3 cm. in 24 hrs. per $cm.^2$ grammes of water	Water taken up by hollow cubes in per cent. of weight of the cube		
				1 hour	24 hours	24 hours	8 days		24 hours	8 days	Note
1:2·5	40	563	242	2	5	83	81	88	2·5	4·0	Wet through, water has entered
			212	0	0	85	85	0	0	0	Perfectly dry

It was striking, and, indeed, surprising, to find, in the case of these soft-soap test specimens, not only that the hollow cubes and the prisms were completely waterproof, but also that the disc 1 cm. in thickness could maintain a water column of 85 cm. which had not sunk after 8 days. Still more striking is the experiment in which the disc of 3 cm. thickness withstood the pressure of 2·1 atmospheres without showing any permeation of moisture after 24 hours.

This experiment demonstrates in a brilliant fashion that a dense concrete which has been prepared with soft soap will be completely impermeable to water, without our having to go through the laborious operations required for a successful application of fluates.

It is not only this experiment which proves the correctness of the facts that have been ascertained. Practical experience confirms this experiment. The tank of the water tower on Margaret Isle in Budapest, 600 m.3 in capacity, was prepared with potash soap, after these preliminary experiments with hollow cubes had proved the concrete which was used to be completely impermeable to water. The preparation of such concrete with the potash soap is not expensive. The cubic metre of concrete would at the highest be more expensive by 10 kronen (8s. 6d.).

It is true that a firm and quite dense concrete, free of voids, which has been most carefully prepared and faced with cement, will also be waterproof; but this object can be attained by the aid of potash soap with much less trouble and care at much smaller expense.

One important question remained to be inquired into. It must be admitted that a considerable saving might be effected if the same object could be obtained by using much smaller quantities of cement. The question was: Is it really absolutely necessary that the cubic metre of concrete should contain 500 kg. of cement? Might not the aim be secured with 300 kg. of cement?

I therefore conducted some experiments in which the cubic metre of finished concrete contained only 300 kg. of cement. For these tests I prepared hollow cubes of 20 cm. of edge, the wall thickness being 4 cm. The specimens were made with Portland cement, which after 28 days yielded:

Tensile strength, 34·2 kg.; crushing strength, 337 kg.

The Danube sand used for these experiments had a density of 1·62, and the volume of the pores amounted to 35 per cent. The coarse additions consisted of Danube pebbles of 2 cm. diameter at the highest, a density of 1·59, and a pore volume of 39 per cent. The concrete was made with 71·5 per cent. of water and 8 per cent. of soap solution (calculated for the cement applied), and was kept damp for three weeks before being submitted to the test. We also made from this concrete prisms of 21 cm. edge length and 5 by 5 cm. cross section. These were likewise kept in the damp for three weeks and afterwards dried in the stove at 100° C., and then placed in 10 cm. of water in order to determine the amount of water taken up by the specimen.

The results are tabulated below. We should add that we also prepared cubes of 20 cm. edge for the pressure test, and that the resulting strengths tabulated are means of three experiments.

Mixture ratio by weight	Liquid added	Amount of liquid in per cent. of cement	Cement in kg. per m³ of mortar	Crushing strength after 28 days	Sinking of the level of the 2 l. of water in cube 150 mm. water depth after				Weight of prism after standing in 10 cm. of water, grammes			Water pressure of 2·1 atm. forces through disc of 3 cm. thickness after 6 days per dm.² grammes of water	Note
					1 day	2 days	1 week	2 weeks	originally	1 day	1 week		
1:1·5:2 5	Water	71·5	300	83·2	10	15	42	65	1162	1223	1245	38·4	The discs were dried after 3 wks.
	Soap		294	75·0	1	1·5	2	3	1176	1184	1194	0	

These experiments demonstrate that a waterproof concrete can also be obtained by the aid of a potash soap if 1 cm.³ of concrete does not contain more than 300 kg. of Portland cement. Just like the previous tests with 500 kg. of cement, these tests demonstrated that the discs 3 cm. in thickness, when exposed to a water-pressure of 2·1 atmospheres for 6 hours, did not permit the smallest quantity of water to permeate; they did not even turn damp when potash soap had been used in their preparation. The same discs were completely wetted through in one hour when they had been prepared with water.

When a waterproof concrete is to be made with potash soap, the chief condition to be observed is that the concrete should be perfectly dense—that is to say, free of voids. The pores between the sand particles should be entirely filled with cement; but since this would require a great deal of cement, it is important to know that the mortar may be mixed with pebbles and crushed stones, such that part of the volume of the pores is filled by this material, which is embedded in the mortar.

SUMMARY.

1. A waterproof concrete can be prepared by the application of fluates. The operation, however, requires a great deal of time and labour.

2. By the application of an 8 per cent. solution of potash soap, instead of water, in mixing, the concrete can be rendered waterproof, so as to fulfil all requirements as to impermeability to water.

3. In order to prepare waterproof concrete with the aid of potash soap, we have to apply at least 300 kg. of a good Portland cement per cubic metre of concrete. When the cement is of a less superior quality it will be advisable to take 500 kg. of cement per cubic metre of concrete.

The author comes to the conclusion that a waterproof concrete which will satisfy all requirements can be obtained with the aid of a potash soap solution of 8 per cent. when from 300 to 500 kg. of Portland cement are used per cubic metre of concrete.

FUNDAMENTAL PRINCIPLES FOR THE ORGANISATION OF THE PUBLIC TESTING OF MATERIALS.

By A. MARTENS, Director of the Royal Prussian Government Institute for Testing Materials, Gross-Lichterfelde, near Berlin.

WITHIN the last thirty years the testing of materials has acquired a permanent position among Government institutions, and at the present time the subject has been very fully developed by a large number of countries, whilst others are endeavouring to raise it to a high status.

In speaking now of the organisation of such testing institutions, it is necessary for me to premise that this organisation must differ according to the local conditions in each country; but as these conditions are beyond my cognisance and I therefore cannot deal with them, I shall confine myself chiefly to those which prevailed in the case of Gross-Lichterfelde. Even these, however, I shall merely touch upon, since in my opinion they are by no means ideal, but are capable of improvement in many respects.

In the first place we will draft the fundamental principles for the establishment of a public institute for testing materials, and then proceed with the plan of such an institute on a large scale.

A well-equipped institute for testing materials can contribute in a very important degree to the furtherance of the industry and commerce of a country, provided it meets the requirements of the consumers and producers of materials, without being subjected to one-sided influence from these circles in their own interests. Consequently, whilst maintained by the State, such institute should be exempt from the influence of interested State technical officials (railway, traffic, military, etc., departments) as also from that of other interested persons. Its sole obligation should be to test materials, at the request of any citizen, freely in accordance with the laws of scientific technology and with all skill, by the aid of highly reliable instruments, whose accuracy is being continually checked, and to act as arbitrator, at the request of the parties concerned, in case of disputes.

Although, according to this, the institute can be called in aid by anyone, it is nevertheless unnecessary that it should perform its functions gratis. On the contrary, it seems perfectly just that the parties who have derived benefit from the work performed should be called upon to pay a suitable charge, sufficient, at least, to cover the outlay on labour and testing appliances. It may be left an open question whether, in addition, the outlay on administration expenses, cost of site and plant, should be charged in full to the parties in question. In my own opinion, this portion of the expenses should be borne by the State Treasury because, as already mentioned, one is justified in requiring that a State-supported testing institute shall be managed in accordance with all the rules of art and science and serve science free of charge. More especially may one require that the institute shall of itself (that is to say at the cost of the State) aid science, at least in all the branches which need to be developed for the purpose of completely solving its own problems. The inevitable consequence of this will be that the institute will also have to perform such tasks as will afford general assistance to the trade and industry of the country, and which therefore fall to the charge of the public, *i.e.*, the State. This, however, does not preclude the possibility that, in special instances, large associated interests may, at their joint expense, employ the services of the institute for extensive tasks. In this way the Gross-Lichterfelde testing institute has, in fact, carried out very extensive tasks in common with certain industrial groups and technical associations in Germany.

Even in these circumstances the tasks and performances of the institute must be estimated on a purely commercial basis. Accordingly, it must be strongly insisted that not only should the individual citizen pay for the work which has been done for him directly, but also that the expenses incurred in the public interest shall be covered by an income corresponding to the ideal services performed for the commonweal. The extent of this value, however, cannot be estimated by the Minister of Finance, but must be established by a committee of experts, who can at the same time form a permanent advisory board for the management of the institute, and must be composed and organised in such a way that all one-sided interests are excluded.

On this account, the advisory board should be an honorary body, appointed for a limited time, and on which high administrative officials in the State service will sit in association with representatives of technical bodies, technical officials and representatives of commerce and industry; so that, whilst all interested circles have a voice on the board, the institute cannot be exposed to any one-sided influence. The managing officers of the institute should have seats on the board, so that they may be personally responsible, and can also represent the interests of the management personally on the board.

If, as is the case at Gross-Lichterfelde, the institute has to deal with the entire domain of the testing of technical materials, it must be divided into a number of departments, each under the management of a very active specialist, so that in each case the experience gained in that particular branch may be represented very completely. These managers must dispose of an adequate staff consisting of men experienced in the branch with which their department is concerned, and other assistants. It is advisable to give the higher officials permanent positions, but to make the appointments of the assistants revocable, in order to ensure strenuous activity and facilitate the weeding out of unsuitable elements.

The total equipment of the institute should be arranged, by grouping the individual departments, in such a manner that the different rooms are in the most suitable juxtaposition, and can be supplied with power, light, heat, etc., from a central generating station. There should also be central offices for internal communication (telephone), the administration of the domestic concerns of the institute (house, outdoor and garden), purveying, control of materials, library, collections, etc.

The supervision of the whole should be confided to a directorate, charged with the external representation of the institute and the supervision of the general and scientific work, whereas intercourse with those who desire to have work done should be preferably in the hands of the departmental chiefs, this being, of course, carried on in accordance with certain general principles common to the whole establishment.

A sufficient staff and accommodation should be provided for the clerical and cashier's department; and reception rooms, meeting rooms and lecture theatres, fitted with up-to-date equipment, will also be required. A clerical and recording staff will also be required for each department, to deal with the testing certificates, supervision work and scientific results obtained. This is necessary because accurate special knowledge is required for this work.

At Gross-Lichterfelde the general works' management is under the care of the Director, who is assisted in this part of the work by a permanent technical collaborator. The general works department comprises the boiler plant for heating the entire institute and supplying steam to the engines; the electric and hydraulic plant supplying power to the various departments; the general system of conductors and conduits for electricity, gas and water (high-pressure and town-water mains), steam pipes and drains, and also general repairs and the upkeep of the building and appliances.

The institute is divided into six departments, viz.: No. 1 for testing metals, No. 2 for testing building materials, No. 3 for testing paper and textile materials, No. 4 for metallography, No. 5 for general chemistry, and No. 6 for testing oils. Each of these departments carries out all kinds of tests on the materials falling within its scope, and issues reports in answer to questions relating thereto.

SECONDARY STRESSES IN RIVETED JOINTS.

SOME POINTS THAT ARE OFTEN OVERLOOKED IN DESIGN.

By EWART S. ANDREWS, B Sc.Eng. (Lond.)

The subject of Secondary Stresses is one of considerable importance in engineering work, and the following article by Mr. Andrews should be of interest to our readers.—ED.

In designing riveted joints for constructional steelwork very few engineers appear to allow for what are known as "secondary stresses," although such stresses, as we shall show later, often come as high as the primary stresses. Some will argue that the structures designed with the omission of such considerations appear to be safe in practice, so that there is no need to worry about such refinements. But this attitude in the matter is surely not satisfactory if we regard our designing as a science. A similar argument can be, and has been, advanced against making any calculations at all, and this attitude is typical of the mind that considers that what was good enough for our fathers is good enough for us; it is half-brother to the insular conservatism which made the red flag precede the motor-car in this country and which caused reinforced concrete walls to be built thicker than brick walls.

Definition.—We may regard "secondary stresses" as "stresses induced in a structure by a failure of the design to comply with the assumptions that are made in the calculations."

In a calculation for the stresses in framed structures, for instance, we assume that the members are pin-jointed together and that the centroid lines of the various bars intersect at the centres of these pins; we very seldom, however, construct our structure in accordance with these assumptions, and we ought to consider what additional stress this introduces. We shall find that in many cases the results of such considerations are startling.

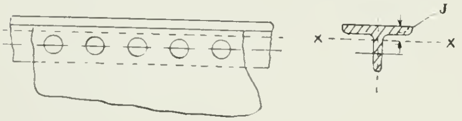


FIG. 1.

Numerical Example.—Take, for a first example, a $4'' \times 4'' \times \frac{1}{2}''$ T-bar used, say, as the rafter of a roof truss.

The centre-line of the rivets connecting this bar to the other parts of the structure is usually at a distance of $2\frac{1}{4}$ in. from the flange.

If the length of rafter between supports be taken as 6 ft. and we treat the ends as pin-jointed, then, according to the usual formulæ—such, for instance, as those tabulated in Dorman, Long & Co.'s "Pocket Companion"—the safe load on this bar is 11·1 tons. Most designers would consider this section as quite satisfactory if the calculated load were not greater than this.

Now, let us look at the properties of this section as given in the standard tables. They are:—

Area $A = 3{\cdot}76$ sq. in.

Distance J to centroid $= 1{\cdot}16$ in.

$I_{xx} = 5{\cdot}40$

In the use of this bar, therefore, the load is transmitted at a distance $=2{\cdot}25-1{\cdot}16=1{\cdot}09$ in. out of centre.

The bending moment due to this is $M=11{\cdot}1\times1{\cdot}09=12{\cdot}1$ in. tons.

To a 1st approximation the compressive bending stress caused in this manner $=\frac{M}{I}y=\frac{12{\cdot}1\times(4-1{\cdot}16)}{5{\cdot}40}=\frac{12{\cdot}1\times2{\cdot}84}{5{\cdot}40}=6{\cdot}4$ tons per sq. in. approximately.

$\therefore$ Total stress $=\frac{W}{A}+\frac{My}{I}=\frac{11{\cdot}1}{3{\cdot}76}+6{\cdot}4=9{\cdot}3$ tons per sq. in.

This is considerably above the "safe" figure.

It will also be noted that the secondary stress of 6·4 tons per sq. in. is more than twice the primary stress of 2·9 tons per sq. in.

A more accurate formula for calculating the secondary stress, which allows for the additional eccentricity due to the deflection of the bar, is—

$$\text{Stress}=\frac{My}{I-\frac{Wl^2}{10E}}=\frac{12{\cdot}1\times2{\cdot}84}{5{\cdot}40-\frac{11{\cdot}1\times72\times72}{10\times13000}}=6{\cdot}9 \text{ tons per sq. in.}$$

E in this formula is, as usual, Young's Modulus.

It is very difficult to say exactly what the actual maximum stress will be in this case, because we cannot get inside the section to find out, but there is, in the writer's opinion, no doubt that it is greater than the primary stress of $\frac{11{\cdot}1}{3{\cdot}76}=2{\cdot}9$ tons per sq. in., which is all that most designers would allow for.

Mr. E. W. Pittman, in a paper read before the Engineering Society of Western Pennsylvania [Proceedings, 1909-10], gives some interesting sketches of riveted joints as commonly designed, and as they should be designed to minimise secondary stresses and to get equal loads on the rivets. Fig. 2 shows a joint in a **N** girder.

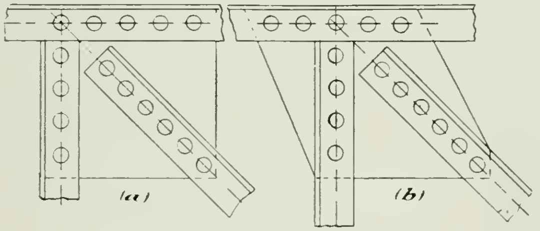


Fig. 2.

(*a*) is the usual arrangement; (*b*) is the more correct variation.

Fig. 3 (*a*) shows a common arrangement for the shoe of a roof truss. This is open to the objection that the centre lines of the rafter and tie do not intersect upon the centre line of the column, so that secondary bending stresses will be caused in the sections, apart from those caused by the eccentricity of the rivets.

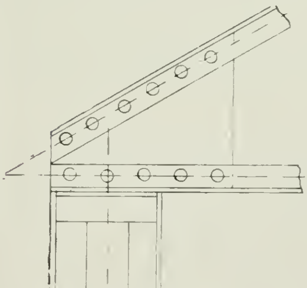


Fig. 3A.

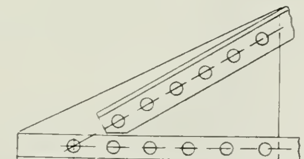


Fig. 3B.

Fig. 3 (*b*) shows a preferable arrangement, also common, in which the above objection is met; but it has, in common with the form above, another objection that might be avoided—viz., that the rivets will not be equally stressed, as their centroid does not coincide with the imaginary pin-joint.

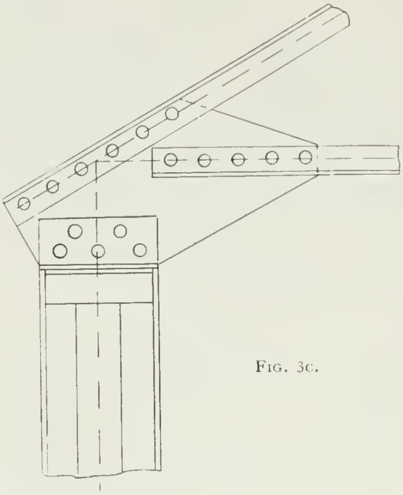


FIG. 3C.

Fig. 3 (*c*) shows an arrangement—not common—in which both objections are avoided.

Fig. 4 shows a knee-brace connection to a column. The form shown as (*a*) is quite common and is open to the objection that the rivets passing through the column, which are in tension as well as shear, are unequally stressed. In the form shown at (*b*) this objection is removed.

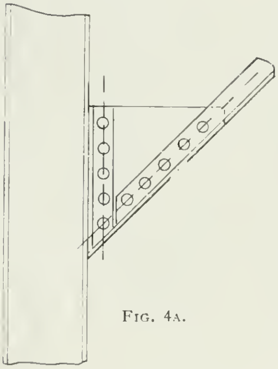


FIG. 4A.

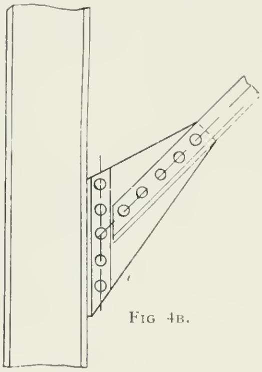


FIG 4B.

Another familiar case in which secondary stresses occur in riveted joints is to be found by a consideration of an angle-cleat for use in connecting **I**-beams together.

Apart from a consideration of secondary stresses, the procedure for finding the number of rivets necessary between the angle-cleat and its beam would be as follows :—

Find the reaction R transmitted from the beam to the other part to which it is to be connected ; calculate also the least strength r of each rivet in shear or bearing

Then the number of rivets necessary $= \frac{R}{r} = n$.

Numerical Example.—Take, for instance, the case of a 16″ × 6″ standard **I**-beam, with a span of 18 ft.

From the tables, section modulus = 90·7.

∴ with stress = 7·5 tons per sq. in.

Safe bending moment = 7·5 × 90·7.

$$\therefore \text{ if load is distributed evenly, } \frac{Wl}{8} = 7{\cdot}5 \times 90{\cdot}7,$$

$$i.e., \; W = \frac{90{\cdot}7 \times 7{\cdot}5 \times 8}{18 \times 12} = 25 \text{ tons, nearly.}$$

$$\therefore \text{ load on each cleat} = R = \frac{25}{2} = 12{\cdot}5 \text{ tons.}$$

Using $\frac{3}{4}$-in. rivets, we have shear strength of each, double shear 5 tons per sq. in. = 5 × 0·884 = 4·42 tons.

Taking web of beam $= {\cdot}55$ in.

$$\text{Bearing strength of each rivet} = \frac{10 \times {\cdot}55 \times 3}{4} = 4{\cdot}12 \text{ tons.}$$

$$\therefore \text{ number of rivets required} = \frac{12{\cdot}5}{4{\cdot}12} = 3.$$

In good practice, however, more than three are used, five being the most common. This is because the load R is distributed along the edge of the cleat (*Fig.* 5), and not down the centre line of the group of rivets, and this eccentricity causes a twisting action on the rivets as a group which increases the stress on each.

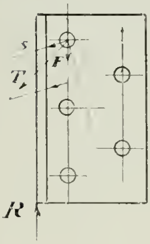


Fig. 5.

The formula for obtaining these additional stresses was explained and proved in an article by the present writer in the "Architects and Builders' Journal" of February 1st, 1911, p. 123.

If x is the perpendicular distance from the load R to the centre of gravity of the group of rivets (*i.e.*, $x =$ eccentricity of load), and s is the distance from any particular rivet to this centre of gravity, then the secondary load S on this rivet is given by—

$$S = \frac{Rxs}{\Sigma s^2}$$

where Σs^2 is the sum of the squares of the distances of all the rivets from their centre of gravity. In the case referred to above (*Fig.* 5) and worked out in detail in the previously-mentioned article, the secondary stress S on the top left-hand rivet for the whole reaction of 12·5 tons comes to 3·18 tons; the primary load F on each rivet is $\frac{12{\cdot}5}{5} = 2{\cdot}5$ tons, $\therefore$ the resultant load T, found by the triangle of forces as shown $= 4{\cdot}40$ tons. This is a little above the safe load of 4·12 tons which we obtained previously.

In the discussion of Mr. Pittman's paper referred to above, Mr. P. S. Whitman gave some very interesting figures of tests on cleat connections.

Each cleat had two rivets on one side—for instance, as given by Messrs. Dorman, Long for 6-in. and 5-in. beams. The results were rather variable, but in most cases the safe load given by the test was between the values for ordinary primary stress calculations and those allowing for secondary stress.

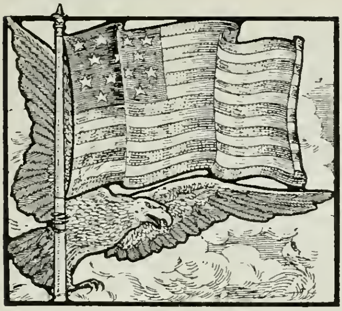
The following results are interesting :—

Test Number.	Safe Loads, in lbs.		
	By Test.	By Ordinary Method.	By New Method.
1	1,000	6,900	1,470
2	6,500	6,900	1,765
3	4,450	13,800	2,940
4	6,250	13,800	3,530
5	1,000	—	—
6	2,000	6,900	1,470

No. 5 is very interesting; the holes in the beam were $\frac{1}{2}$ in. larger in diameter than the rivets, so that the safe load of 1,000 lb. is a measure of the friction exerted by the closing of the rivets; this friction is a very variable quantity in riveted joints and cannot be relied upon with safety for design. Nos. 2 and 4 were riveted in the ordinary way. No. 1 had rough bolts forced in, the nuts being tightened only by hand.

Nos. 3 and 6 had turned bolts, with nuts tightened by hand. It will be noted with interest that in these two cases, which come the nearest to the theoretical conditions of the joints, the results by test agree very much better with the calculations by the new method than with the old.

Although it cannot be said definitely that in design the *whole* of the secondary stresses must be allowed for, it is thought that the foregoing will show that a consideration of such stresses is essential in scientific designing of riveted joints, and that where possible the joints should be arranged so as to minimise these stresses.



THE CONSTRUCTION OF THE ROOSEVELT DAM.

A HIGH MASONRY DAM IN ARIZONA.

We present herewith some particulars regarding the construction of the Roosevelt Dam in Arizona. The work is of special interest on account of the many difficulties that were encountered in the course of erection. We are indebted to Mr. Chester W. Smith, Constructing Engineer of the Dam, for our illustrations, and the particulars were abstracted from an article in the "Engineering Record" by Mr. Chester Wason Smith, Engineer, United States Reclamation Service.—ED.

THE Roosevelt Dam, which was completed last year, has been built by the U.S. Reclamation Service for the purpose of irrigating about 250,000 acres of land in the vicinity of Phœnix, Arizona. It is built across the Salt River, about 60 miles by direct line from Phœnix and about one-fourth mile below the junction of Salt River and Tonto Creek. It forms a reservoir covering 1,329 acres and supplied from a watershed of about 5,800 sq. miles.

The original low-water level of the river at the dam site was 1,903 ft. above sea level.

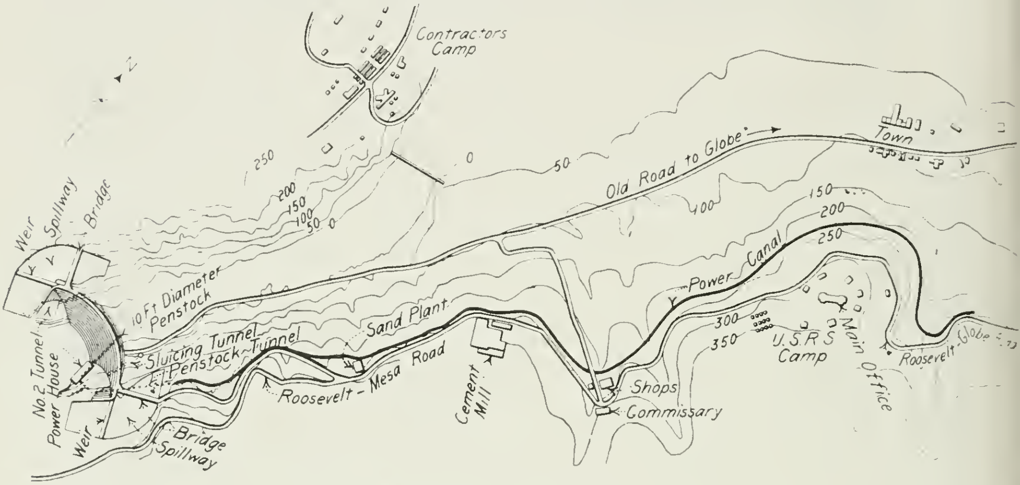
The rock at the site is a very hard, fine-grained sandstone, very distinctly stratified, and dipping upstream at an angle of 29 deg., practically at right angles to the dam. The dam is of gravity section, but is also arched on a radius (to the centre of the roadway on top) of 410 ft.

The parapet walls are 3 ft. 8 in. in height above the roadway. They are corbeled out so as to give a roadway 15 ft. 8 in. in the clear between curbs. The dam proper forms a quarter circle, from each end of which abutments are extended on the tangent. About 35 ft. on these tangents are the curved faces of the wing walls, and from thence there is a waste weir 200 ft. long at each end. A spillway was cut in the solid rock around each end of the dam; from them was taken practically all the stone which went into the masonry. The wing walls above mentioned, curving from the ends of the dam, are for the purpose of forming side walls to the spillways until the latter dip sufficiently below the natural surface.

Across each spillway the roadway is carried on a three-arch reinforced concrete bridge; each span is 59 ft. in the clear. As originally laid out, these bridges were located on the line of the chord joining the ends of the dam, produced each way to the vertical side wall of the spillway. To connect the land end of the bridges on this location with the road would have required either a very expensive undercutting of the rock around the vertical wall, or a reinforced concrete viaduct built around the face of the cliff, with probable danger in the future from the falling of detached rock or boulders. These considerations led to the relocation of the bridges as shown on the accompanying plan, with the square turn onto the bridge eased by cutting off the corner by a reinforced concrete beam and floor.

Through the dam are three openings. A tunnel of 108 ft. cross-section and 450 ft. long was driven through the wall of the canyon around the south end of the dam. This was for the purpose of passing the river while at work on the foundations, and later to serve as a scouring or sluicing tunnel. The inlet of this tunnel is about 125 ft. upstream from the dam, and is screened by a reinforced concrete cage-like tower 78 ft. high.

At the intersection of the tunnel with the upstream face of the dam are placed the gates, six in number, one set being the service gates and the other set, about 10 ft. upstream, being the emergency set, each opening being 4½ ft. by 9 ft. In operation these gates are drawn above the roof of the tunnel by hydraulic cylinders 2 ft. in diameter, in a chamber above. The rods pass through stuffing boxes set in a heavily reinforced concrete floor 7 ft. thick. This operating chamber is at the foot of a shaft built out on the upstream face of the dam, and leading to the top of the dam.



Sketch Map of the Vicinity of the Roosevelt Dam.
THE ROOSEVELT DAM.

The second opening is a 10-ft. diameter riveted steel penstock passing through the dam on the upstream face in the power-house below the dam. The opening on the upstream face is rectangular, about 5 ft. by 20 ft., with a heavy cast-iron grille set in the face of the dam. This opening passes by warped surfaces of concrete to the circular steel penstock. On the upstream face of the dam are grooves for letting down a screen over the opening, or in case of necessity a steel shutter to close it. Hoisting apparatus is provided in a small room just under the roadway. Between the dam and the power-house is placed a balanced valve.

The third opening is near the north end of the dam; it consists of three lines of 5-ft. cast-iron pipes passing from the upstream face through the masonry, and at the junction of the masonry and the natural rock converging into a 9-ft. diameter concrete-lined tunnel, which discharges out of the face of the cliff some 100 ft. below the dam.

Immediately below the dam is the 125-ft. by 38-ft. power-house. Quite extensive and varied work was done preliminary to the building of the dam, including extensive road-making.

Owing to the scarcity of fuel for power purposes it was decided to build a power canal, taking water from the Salt River and conveying it 191 miles to dam site, where a drop of 220 ft. is secured. This canal, which has a capacity of 225 cu. ft. per second, is one of the permanent features of the project, although built primarily for furnishing power to build the dam. It comprises a concrete diversion dam 450 ft. long across Salt River, raising the water 6 ft., concrete headgates, waste gates, culverts, 21 tunnels aggregating two miles in length and all concrete lined; also silt-settling basins. Owing to the nature of the material encountered and the steep side hill, it was necessary to line with concrete the lower $1\frac{1}{2}$ miles of the canal.

Reinforced Concrete Bridge across Spillway.
THE ROOSEVELT DAM.

From the lower end of the power canal a 7-ft. diameter penstock tunnel dips underneath the south spillway to the power-house; its lower half is riveted steel backed with concrete, the upper half being concrete alone.

The contract for the construction of the dam provided that the Reclamation Service should furnish the cement, and tenders were received, the lowest of which, including haul to the work, was $4.89 per barrel. This figure, in connection with the fact that good cement materials existed near the dam site, led to the erection and operation of a cement mill by the Reclamation Service. The mill, of a nominal capacity of 300 barrels per 24 hours, but of an actual capacity of 400 barrels, was in operation early in 1905, and furnished cement to finish up the power canal. It was a dry process mill, using crude oil for fuel. The oil cost $3.48 per barrel of 42 gal., delivered in Roosevelt, being hauled in tank wagons 60 miles from Mesa. One barrel of oil burned four barrels of cement. During the five years the mill was in operation it turned out about 300,000 barrels of first-class cement, at a cost, including plant and office charges, of about $2 per barrel less than the tender above mentioned, thus amply justifying the course of its builders. This cost would have been even less could the mill have been operated continuously at full capacity.

Two 2¼-in. diameter Lidgerwood cableways about 1,200 ft. long span the river, one on the line joining the ends of the dam, the other about 85 ft. upstream from the first. These transfer the rock from the quarries at either end to the derricks on the dam, and also the concrete and mortar from the mixing plant. The mixing plant was situated at a point about 125 ft. upstream from the dam on the south side of the river. The mixing plant was one terminal and the cement mill was the other of a Laschen aerial tramway 1,700 ft. long for carrying cement and sand to the mixer. Below the bins was a Smith mixer, mixing 1·3 cu. yd. of concrete per batch; two batches were dumped into a 3½-cu. yd. box and rolled out under the cableways.

Reinforced Concrete Bridge across Spillway.

THE ROOSEVELT DAM.

During 1905 the contractors were engaged in establishing camp, getting in and erecting plant, stripping quarries, and starting works for diverting the river. The contract required temporary dams above and below the excavation. The contract also required the construction of a flume 50 ft. wide, with its bottom 5 ft. below the top of the dam, for the purpose of handling a higher stage of the river.

When the contractor was well along with the driving of piles for the temporary works there occurred one of the largest floods in the history of the river.

This took out every vestige of work done in the river, and also some of the machinery. The river continued so high that nothing more could be done in the bottom for some months, and during that time it was decided, upon the request of the contractors, to eliminate the flume altogether, as being very difficult, if not impossible, to build in such a manner that it would withstand flood. In other words, it was decided to proceed on the basis of having the pit accessible for work only when the flow of the river was less than 1,300 sec.-ft.

During March, April and May they worked on the coffer-dams and on June 13th diverted the river through the sluicing tunnel; for the next three months the work was making the cofferdams tight, excavating loose material and rock, and preparing the foundation.

As for the rock found in the bottom, its surface corresponded very closely in elevation to the indications of the preliminary borings, and being in a stream bed where the process of erosion was very active, it was generally sound, rendering but a small amount of excavation necessary.

On September 20th, 1906, the first stone was laid in the dam. From this time on to December 2nd progress was continuous, but on the latter date a flood stopped work, practically destroyed the cofferdams, and prevented further work till the following spring.

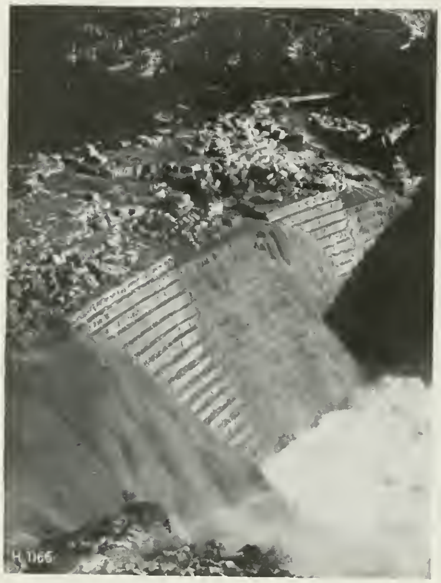
In May, 1907, repairs to cofferdams were completed, the river again diverted through the tunnel and excavation started in the pit, which, of course, had been filled. On June 18th the laying of masonry was resumed, but, on account of floods similar to that of July 24th, the work could be carried on only about half of the time until early in November. The months of November and December, 1907, and January, 1908, were free from any interruptions, and work was prosecuted vigorously.

The masonry was first built for the upstream third of the dam and carried across the river, thus cutting off all leakage from the upstream side. The masonry for the downstream, two-thirds in thickness, was then built across, beginning on the south side and carrying it to the same elevation.

By February 1st, 1908, the bottom was all in. All danger of serious interruption from floods had been passed, the cofferdams were no longer required, and no more pumping was necessary. Preparations were being made to instal the gates and operating mechanism, and for this purpose the entrance to the sluicing tunnel was blocked, and the river all diverted through the channel left across the north end of the masonry. Construction went on in the meantime on the south two-thirds. Early in June the work on the gates had reached such a point that the river was again sent through the tunnel and work resumed on the north end of the masonry.

During the construction of the dam, until all danger from floods had passed, the north end of the masonry was kept considerably lower than the rest, in order that any water going over the dam would not interfere with the power-house. In December, 1908, a flood came along. The body of water falling tore out some rock at the toe of the dam. Soundings, taken after the water stopped flowing over, compared with two lines of the original borings, gave an estimate of 12,000 cu. yd. thus taken out. This was principally from a stratum of black shale, which cut across the river immediately below the toe of the dam, and was more easily disintegrated than the rock under the dam.

Early in May, 1909, an opportunity was taken to close the tunnel for a day and look at the gates. Inspection showed that the four or five months' discharge of the river through the tunnel, at heads from 50 ft. to 90 ft., had made necessary some repairs and alterations to the gates, to the concrete piers between them, and to the tunnel lining immediately below. Not to enter into too much detail, it may be said that, while the main structure of the gates was not threatened, some accessories and details had been entirely torn out; the downstream end of the reinforced concrete piers between the gates, as well as much of the concrete lining for 20 ft. or so, was practically demolished.



Flood of December, 1908.
THE ROOSEVELT DAM.

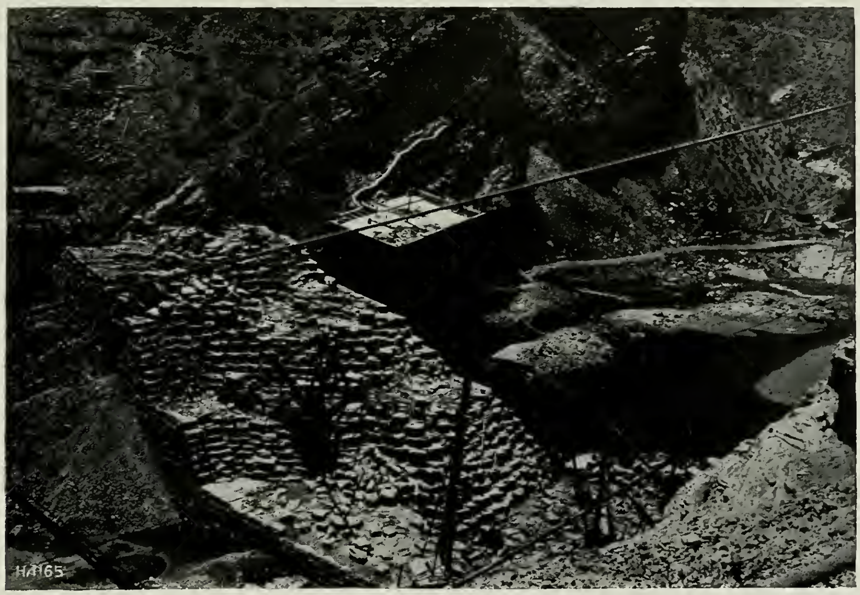
Besides alterations and repairs to the service gates, the tunnel, floor, sides and roof for a distance of 30 ft. from the gates was lined with heavy steel plate frequently and firmly anchored to the rock and made solid by pumping grout in behind them. The downstream ends of the piers between the gates were rebuilt and faced in the same manner, the side walls of the piers between the two sets of gates having been originally constructed of heavy castings. Coincident with these repairs, and owing to the conditions which made the repairs necessary, tunnel 2, previously described as the third opening through the dam, was constructed.

It was not till November, 1909, that the work in the tunnel was completed so that the reservoir could be pulled down to allow the gap in the masonry to be closed. Work on the remainder of the dam had proceeded, of course, during the season.

During May and June, 1910, a small cofferdam was built between the power-house and the pool below the dam, and the pool pumped out for the purpose of seeing the effect of the flood of December, 1908, and making repairs if necessary. Some undercutting of the masonry was done, the maximum being about 4 ft. The masonry was perfectly intact, and the rock underneath was sound and tight, no water coming through or under the dam. The rock faces left were worn quite smooth.

Something over a thousand yards of masonry was built in the form of a buttress and coming up several feet outside the toe of the dam. The illustration on page 828 shows the dam approaching the top, with the stagings erected on the steps of the masonry to serve as landings for material and to support one angle of the derricks. It also shows a discharge from tunnel 2 on the occasion of the trial operation of the balanced valves. A 2-in. pipe, with a small hole each 10 ft., conveys water under gravity from the power canal and covers the face with a thin film. The very rapid evaporation in this country, hardly to be appreciated by one not familiar with it, should keep the face temperature of the masonry many degrees, quite possibly 75 deg., below what it would otherwise reach, which, it need hardly be pointed out, is a consummation devoutly to be wished. Provision is being made to pump the small quantity of water necessary to keep wet the top of the dam and whatever of the upstream face may be exposed.

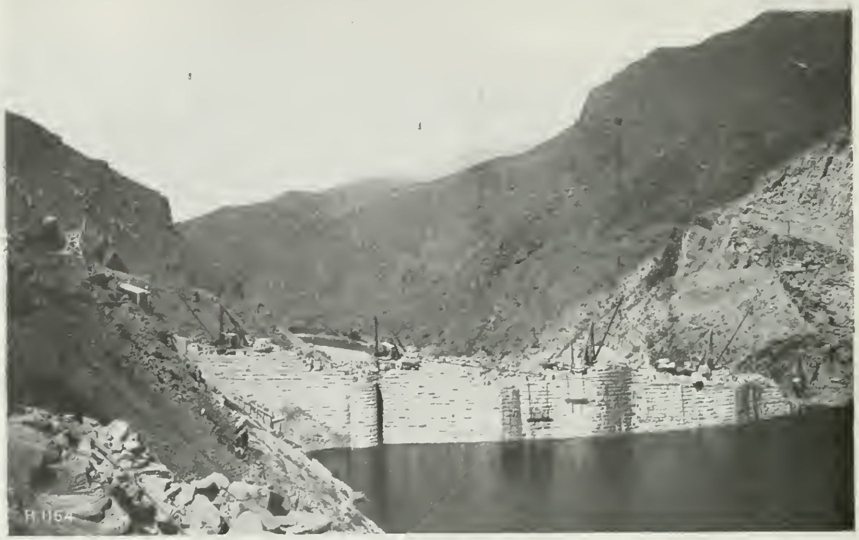
The illustration in our frontispiece shows conditions early in October, 1910, with the dam practically completed. One derrick is engaged in laying the roadway and the parapet walls; the bridge at the right is completed, except for the pipe railing; the bridge at the left is practically as far advanced, except that the forms are still in place. There remained to be done a little masonry in the north wing wall, a little in the crests of the waste weir across each spillway, and a small amount of excavation at the downstream end of the south spillway.



Condition of Dam on June 30, 1908.
The Roosevelt Dam.

The dam, wing walls and waste weirs complete (not including bridges) contain 342,500 cu. yd. of masonry, and barring the items of parapet wall and the repairs at the toe, the entire mass has been built at the rate of 13·2 cu. yd. for each hour of derrick actually engaged. This rate per derrick-hour includes the start in the bottom and the construction of the thin part at the top, in both of which the rate was much below the average. The first 263,000 cu. yd. were laid at the rate of 15·3 cu. yd. per derrick-hour, and the rate for many months was 16 to 18 cu. yd.

Probably this rate has never previously been attained on a similar structure. It was due to the manner of laying the masonry, the vertical joints being filled with quite wet concrete, largely placed by dumping, then churned and spaded when necessary, and spalls thrown in. Compare this manner and rate with that of some other masonry dams where the classic method prevailed of filling the vertical joints with comparatively stiff mortar and spalls laboriously laid up by a mason with a trowel. It may be further said that the new method results in masonry more nearly watertight than the old.



Upstream Face of the Dam on November 1, 1909.

THE ROOSEVELT DAM.

The masonry is composed of : Mortar, 14 per cent. ; concrete, 36 per cent. ; spalls, 10 per cent., and large stone, 40 per cent. One cu. yd. of masonry required, on an average, ·76 barrels of cement.

Mr. F. H. Newell is director of the United States Reclamation Service; Mr. Arthur P. Davis, chief engineer; Mr. Louis C. Hill, supervising engineer for the South-West; Mr. F. P. Teichman was the designing engineer, and Mr. Chester Wason Smith was the constructing engineer on the Roosevelt Dam.

Fire-Resisting Slab Partitions: Their Uses in Modern Buildings.

By HAROLD G. HOLT, A.R.I.B.A.

In our January number we published an article by Mr. Holt on "Fire-Resisting Concrete." We now publish an article on "Fire-Resisting Slab Partitions," which should be of particular interest in view of the increasing attention given to fire-resisting construction.—ED.

In view of the rapidly increasing use of various patent partitions, which are claimed to be "fireproof," "sound-proof," and "everything-else proof," some inquiry into the various uses and possible uses for partitions of a reliable character will be of interest.

The advantages of the best types of thin fire-resisting partitions are well known; the requirements of such types of partition as we are considering may be briefly summed up as follows:

1. *Economy of space and material; ergo* economy in first cost, weight, and ground space.
2. *Fire-resistance* of greater or less character according to nature and thickness of partition.
3. *Sanitary and vermin-proof qualities.*
4. *Non-conductivity of sound, heat, and cold.*
5. *Mobility, rapidity of erection, and drying.*

Provided that the above requirements be complied with, there accrue considerable advantages in the use of patent partitions.

The architect called upon to select a partition finds, perhaps, some difficulty in the matter; the adoption of any particular partition in a building of much importance is a matter of some moment. Many manufacturers claim the above special advantages for their own make, and the great demand for partitions fulfilling these requirements has led to numerous varieties being placed on the market, some of which do not by any means come up to standard.

The function of a partition is to separate one room from another, or to divide a large room into several smaller ones; of chief importance, apart from question of cost, is therefore the fire resistance of the partition adopted, having due regard to the fire exposure to which it may be subjected. There is no doubt that the time is coming, and coming rapidly, when such undesirable features as wooden-studded and matchboarded partitions, along with other inflammable material in floors and finishings, will be things of the past, and a contributing factor to this is the increasing cost of timber suitable for building purposes. Not only in offices, warehouses, public buildings, and institutions, but even in small domestic houses there is no doubt that timber construction should be, and will be, eliminated as far as possible. In certain cases wood finishes and trims, such as wainscot panelling, etc., will continue to be used for æsthetic reasons alone, but, granted the desirability of the latter, the fact remains that such trimmings add greatly to the fire hazard of any room in which they are found. Fire-resisting construction in the shape of well-protected skeleton members, floors, and so forth, has reached a fairly high state of perfection; given sufficient funds at the architect's disposal, buildings can now be erected which will successfully resist fierce fire for many hours without serious damage to the integral parts of the structure.

With the contents of the building the case is different; these are nearly always inflammable, and in the ordinary type of modern building the problem is to effectively cut up the fire risk into numerous small compartments in which, though the contents be ablaze, the fire is yet limited to a comparatively small area.

Right here the utilisation of fire-resisting thin concrete, or other satisfactory partition, becomes essential. Brick walls do undoubtedly form the most efficient fire stop against a large cubic extent, and are demanded in certain buildings, and in a reinforced concrete building the advantages of a reinforced concrete fire stop wall will in some cases outweigh those of thin walls consisting of small units. In every other case, it may be said, special fire-resisting partitions are necessary. Such may be classed in the following varieties:

1. Concrete slab, reinforced or otherwise.
2. Plaster slab, do.
3. Porous terra-cotta.
4. Porous brick.
5. Metal sheeting and plaster or concrete.

The first four consist of separate slab or block units, built up with mortar joints; the latter consists of a more or less homogeneous mass built round the metal, which is first erected. I propose to deal chiefly with the first four kinds. For purposes of determining the value of such partitions for fire resistance the following standard has been arrived at, and the official tests of the British Fire Prevention Committee show the standard fire resistance of the various partitions tested.

STANDARD TABLE FOR FIRE-RESISTING PARTITIONS WITH OR WITHOUT FRAMES.

Classification	Sub-Class	Duration of Test	Minimum Temperature	Thickness of Material	Minimum Superficial Area under Test	Minimum Time for Application of Water under Pressure
	Class	Mins.	Degs. Fahr.		sq. ft.	Mins.
Temporary Protection	A	45	1,500	2 in. and under	80	2
	B	60	1,500	optional	80	2
Partial Protection	A	90	1,800	$2\frac{1}{2}$ in. and under	80	2
	B	120	1,800	optional	80	2
Full Protection	A	150	1,800	$2\frac{1}{2}$ in. and under	80	2
	B	240	1,800	optional	80	5

Several interesting and instructive tests have been conducted by the British Fire Prevention Committee on partitions erected by various makers, some account of which is given below. It should be noted that the Committee, recognising that special care in erection is often given in the test cases, conduct the latter under circumstances which they consider somewhat more severe than would obtain in actual fires, so that the results approximate to conditions of actual practice.

One of the earlier tests was conducted on a partition form with "Kulm" Partition Slabs, erected by Messrs. H. W. Cullum & Co., London, with a view to obtaining classification as "Partial Protection," Class B. The slabs used in this test were 21 in. long by 11⅝ in. by 2¾ in., weighing 14 lb. per sq. ft., and having horizontal semi-circular tongues and grooves, and square vertical joints. The material of which they were composed consisted of pumice stone, volcanic sand, and Portland cement; the joints were bonded and made in plaster of Paris and fire-clay 1 to 1, and the partition was plastered on the exposed side with plaster composed of fire-clay, plaster of Paris, and sand, averaging ⅜ in. thick. On the unexposed side the joints were merely brushed over with a wash of plaster of Paris, sand, and fire-clay, no plastering being applied; the total thickness of the partition was therefore 3⅛ in., it was built across the Committee's Hut No. 2, 10 ft. wide, and was 9 ft. in height, the area tested thus being 90 ft. super. The time allowed for drying was twelve days, the partition was erected on July 14th, 15th, and 16th, and tested on July 28th, 1904; coke fires for drying purposes were lighted on July 25th and kept burning till the 27th. Readings taken at two points during the test, which lasted for two hours, showed that temperatures ranged on the fire side of the partition from 845° F. to 2,010° F. at the end of the test, a temperature of 1,800° F. being attained at the end of 70 minutes. Water application for two minutes ensued. As a result of the test the partition showed several horizontal and vertical cracks and about one-fourth of the plastering was washed off by water; the surface of the slabs where this occurred was eroded for about ¾ in. Generally, the partition stood up well and showed that such a construction would serve as an efficient fire stop under similar circumstances, but would have to be renewed afterwards; this is quite as much as can be expected of any material or system of construction exposed to such high temperatures for the same period of time; in fact, all experience of fierce fires lead inevitably to the conclusion that the most logical method of effecting the fire resistance of any structural member is to afford it protection for as long a time as possible—sufficient to allow either of the burning out of the fire or its extinction by various methods—by a "secondary" protection which is capable of withstanding the combined effects of fire and water for such a period, but which, in itself, will be probably so much affected that it will require renewal in order to again give the requisite protection to its relative member. Especially will the latter be the case if water under pressure be brought to bear upon it during the course of the fire extinction.

That such renewal should be capable of being effected in the easiest possible manner and without any damage to the integral portion of the structure goes without saying.

Another partition tested on July 28th, 1904, was constructed of *Brunckhorst Partition Slabs*, erected by Messrs. H. Gust & Co., London. This was also with a view to obtaining classification as "Partial Protection." The partition was formed with slabs composed of plaster of Paris and coke breeze in equal proportions, each slab 12 in. high, 2¼ in. thick, having lengths varying from 10 in. to 2 ft., and with vertical and horizontal joints grooved and serrated. The weight was 80 lb. per sq. yard. In the centre of the joints both vertical and horizontal iron wires, 1/16 in. diameter, were placed, the former were secured to hooks in the walls of the testing chamber, whilst the latter were twisted over them in double thickness. All joints were pointed up with plaster of Paris and coke dust in equal parts, and the exposed side of the partition was plastered with the same mixture, average ¼ in. thick, and a wash of the plaster mixture was applied to the unexposed face.

The partition was erected on June 29th and 30th, and a drying coke fire was lighted on July 25th and kept burning until the 28th, the day of the test. The test was for 90 minutes with temperatures ranging from 1,180° F. to 1,800° F. on the fire side, and not more than 126° F. on the unexposed side. Water at 40 lb. per sq. in. pressure was turned on the fire side for two minutes after the gas was turned off. Several vertical and horizontal cracks appeared in the partition during test on the unexposed side, the plastering was badly cracked at the end of one hour, and flaking off of the plaster was general over the face of the partition; on the application of water part of the surface of the slabs was washed off and a hole, 6 in. by 3 in., was made through the partition at one of the joints. Observations made seven days after the test showed that the unexposed face remained unchanged since test, the exposed face of the slabs not eroded by water were intact; the wiring was exposed in places. Neither fire nor smoke passed through the partition.

A further test on concrete thin partitions was that conducted on a partition of *Phœnix Fire-Resisting Slabs,* erected by Messrs. Van der Vygh Bros., Amsterdam and London. In this test the slabs used were composed of pumice stone 20 parts, slag sand 10 parts, hydraulic lime 18 parts, tan (spent or unspent) 12 parts, plaster of Paris 20 parts, water 20 parts.

The slabs measured from 1 ft. 3 in. to 3 ft. 11½ in. in length, 11 in. in height, and 2¾ in. in thickness, and had two vertical grooves sawn out at each end for jointing, and also grooves running diagonally across the horizontal edges. During erection wood slips were used to keep the slabs apart half an inch; these were subsequently removed and iron pins, 11 in. to 12 in. long, 5/16 in. diameter, spaced 16 in. apart, were used to secure the courses together. Three days, June 12th to 14th, 1903, were occupied in finishing the partition, the joints were filled in with one part of Portland cement to three parts of sand, the inside face of partition was plastered one coat coarse stuff of one part cement to four parts sand, and setting coat applied afterwards, composed of one part cement to three parts sand. The outer face of partition was plastered one part cement to five parts sand. Both faces of the partition slabs were keyed for plaster. The total thickness of the finished partition was 3½ in., ½ in. plaster on exposed face, and ¼ in. plaster on unexposed face. On July 1st a coke drying fire was lighted and kept in until the morning of July 8th, 1903, the day of the test. This was for 75 minutes, followed by two minutes' water application at 40 lb. pressure. Temperatures from 995° F. to 1,940° F. were observed on the fire side; the maximum temperature on the outside face did not exceed 125° F. at any time. The effect of the test was that the setting coat of plaster on fire side remained in position, but part of the rendering on fire side was struck off by water. Cracks following the joints were the only effects observed on the unexposed face, but the plastering on the inner face was greatly washed off by water and the remainder disintegrated. Face of slabs on fire side was disintegrated and sodden by water. No fire passed through the partition. The area tested was 87·5 ft. super.

Referring to the second type of partition—*i.e.*, plaster slab partitions—we find a test on the "*Mack" Patent Slabs,* by Messrs. J. A. King & Co., London. These slabs are formed in a somewhat similar manner to the old "plaster floors" with which I have met in the Midlands and which will be no doubt familiar to some of my readers who have had to deal with old buildings there. Such floors were generally constructed between stout oak joists, and consisted of thick plaster in which reeds and rushes were embedded—interesting as an early attempt at fire resistance.

The "Mack" slabs used in the test by the British Fire Prevention Committee were made of plaster, cocoanut fibre, cork dust, and thirty to forty reeds running horizontally along the length of the slab, which varied from 1 ft. 6 in. to 4 ft. 10½ in.; the slabs were 2⅜ in. thick and 10 in. in height.

Vertical V joints were sawn out of the slab ends; the slabs were convex shaped on top and concave at the bottom, somewhat similarly to the "book-tile" used in American buildings. One face of the slabs was keyed for plaster, two coats of which were applied to the inside face, 1 in. thick; one coat only was applied to the unexposed face, ¼ in. thick; total thickness of partition, 3⅝ in. The vertical double V joints were grouted with, and the slab courses set with, mortar composed of plaster of Paris 2, lime 1, and sand 2 parts.

The test was undertaken on May 23rd, 1900, the construction of partition having commenced on May 1st. The maximum temperature during test was 2,050° F., the heat test lasted for 1 hour 10 minutes, and water application for 2 minutes at 20 lb. pressure followed. Fire did not pass through the partition, which remained complete; the plastering on the exposed side was cracked and disintegrated and a portion fell off on the water application. Examination after test showed that the exposed face of slabs was disintegrated for a depth of about ¾ in. and the inner row of reeds was charred.

A *Porous Terra-cotta Hollow Tile Partition* was tested by the British Fire Prevention Committee on August 16th, 1905. This was formed of 2⅛ in. tiles, erected by the National Fireproofing Co., Pittsburg, U.S.A. The latter are large American manufacturers of hollow tiles for partition, floor, roof, and column construction. The terra-cotta is made from various clays mixed with sawdust and burnt at high temperature, the sawdust being consumed, thus giving the required degree of porosity. The partition in question was built of tiles 12 in. by 12 in., average 2⅛ in. thick, with both faces ridged for plaster, and with three through vertical perforations divided by webs forming the hollow portions. The weight of each tile was 12 lb. to 12½ lb. Including plaster, the total thickness was 2½ in. The total duration of test was 153 minutes, including 2 minutes' water application; maximum temperature attained, 1,980° F. on fire side; size of partition, 10 ft. wide, 8 ft. 10 in. high. The effect of test was as follows: Fire, smoke, and water did not pass through the partition, which remained in position, but bulged towards the fire; maximum bulge at centre was 2¼ in., reduced to 2 in. at conclusion of test. Most of the plaster was washed away by water, and several cracks appeared in the face of partition; the face of one tile was split off.

An interesting test on a porous brick partition was conducted on May 4th, 1904. This was on a partition formed with *Jabez Thompson's Patent "Terrawode" Brickwood,* erected by Jabez Thompson, Northwich, Cheshire. This was a thicker partition than any of the preceding ones, and the duration of test was for a longer period—viz., 240 minutes. The blocks approximate to the size of an ordinary brick, and are 9 in. by 4⅜ in. by 3 in. deep. The partition tested was 4⅜ in. thick, unplastered, and pointed fair face on fire side. The bricks have two slots on one face, and this face was away from the fire side; there is one slot in the end of eack brick for the joint. The partition tested was 10 ft. wide and 9 ft. 1½ in. high. Maximum temperature during test was 2,140° F.; water application for 5 minutes at 55 lb. pressure followed the fire test. Results: Fire did not pass through partition; maximum temperature recorded on unexposed side, 230° F.; no effect caused by water application except flaking off face of five bricks; the partition buckled 1¾ in. at the maximum, the curving being regular from all edges to the centre; several fine cracks appeared, and the mortar was slightly vitrified on the surface, as well as several of the bricks.

Enough has been said to show that partitions of the various kinds mentioned are successful in resisting fire for a considerable time—sufficient, in fact, to enable the contents of any ordinary room to burn out without destroying the partition. The use of such slabs and blocks does not by any means end with internal division walls, though they are chiefly designed for that use, and there are a few points to which I desire to draw attention regarding this. Firstly, with respect to the general fire protection of buildings, it must be borne in mind that all openings in division walls must be of equal fire resistance to that of the partitions if the latter are to be of full service. A common form of door opening in thin partitions is that in which a rebated wooden door-frame is used, the back edge of which is grooved to receive the partition slabs and plaster, the outer face of the frame forming a kind of architrave; a wooden, panelled door is usually hung to these.

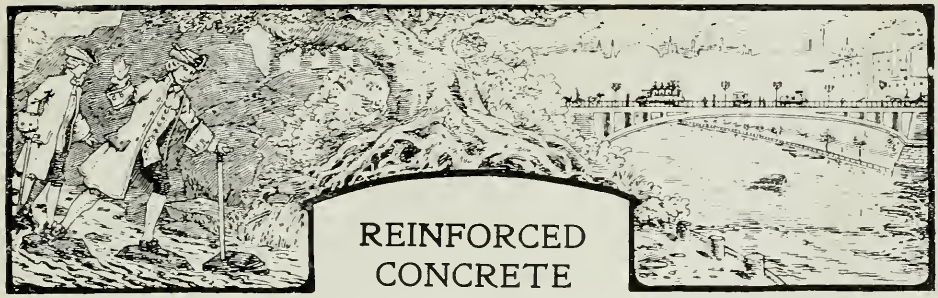
Of course, the fire resistance of such door, if framed up of pine, say, is of little value; and unless the whole of the woodwork be of oak, teak, or other fairly fire-resisting hardwood at least 1¾ in. thick at least, the value of the fire-resistance of the partition is practically lost. Again, any borrowed lights to corridors, etc., should have fire-resisting glazing in fire-resisting frame of iron or hardwood. Doors and trim of stamped steel, such as are used in American office building, will, no doubt, take the place of wooden doors in important public and commercial buildings here; thin doors of concrete specially mixed with a view to fire resistance will also, no doubt, be used in the near future; and partition makers will have to adapt themselves to such changes. A small point, but one to which it would be worth while to pay attention, is the use of partitions in such positions as w.c. and lavatory divisions where they run into a window mullion, one three-light window, say, serving three w.c.'s. In modern buildings of Renaissance or Classic design such a subterfuge is often necessary in order that the fenestration be not interfered with. A partition having special end blocks tapering to 1½ in. in width would often prove a boon to architects who have to put in wider mullions than they desire to cover the width of partition behind. Light roofs may be well constructed with concrete or plaster partition blocks given an outside weather covering. A good field is offered by the use of partitions for the economical construction of small factory and warehouse roofs, particularly of the one-storey shed type. Especially is slab construction suited to covering light steel roofs which require exterior and interior protection. The "southern" slope of north light or saw-tooth roofs may be very suitably built with light concrete or other fire-resisting slabs, and, provided that fire-resisting glazing be used in the skylight area, the result is highly satisfactory from a fire-preventive standpoint; such roofs are often constructed of timber, slates, and plastered below, and are particularly liable to take fire from sparks from locomotives, burning timber, and sparks from higher buildings, etc. This shed type of roof is now often economically constructed on the "warren girder" principle, leaving the top steel boom exposed and the intermediate braces following the roof outline. This gives a tremendous uninterrupted floor area, since very few columns are required. The protection of the exposed boom from fire and weather can be easily effected with slabs and concrete covered with cement or asphalte. Roofs of projecting shopfronts must be constructed of fire-resisting material under the London Building Act, and slab fire-resisting work is here again very suitable. Many roofs are erected in America with terra-cottta tiles on this principle, and they are capable of receiving a variety of finishes, both weatherproof and fire resisting. For the additional fire protection of existing buildings much may be accomplished by the judicious introduction of fire-resisting slab partitions, particularly in view of cutting off such hazards as staircases, hoist wells, chutes, etc. Wooden floors may be afforded protection on the underside, which is subject to the greatest heat during fire, by means of special shaped blocks fitted to the joists and carrying slabs of fire-resisting material; pugging with slag wool or similar material would add to the fire resistance. Staircase bulkheads and bulkheads over basement windows in warehouses and shop stallboards are all danger points from fire hazard passing from one floor to another, and fire-resisting partition blocks offer a ready means of constructing such details in a satisfactory manner.

Exterior walls may in many instances be economically constructed with a light steel framework carrying the main floor and roof loads fitted round with fire-resisting slabs. The garden city movement offers a good field for the ingenuity of partition makers in the construction of cheap cottages which may be finished externally with rough-cast. In very damp situations such would be unsuitable, but in the South and Midlands many small houses and cottages have been built on these lines and offer good insulation against heat and cold. Buildings of a temporary nature, such as exhibition buildings, cinema theatres, etc., may be very cheaply built of slab and steel construction, and offer great advantage in the way of fire resistance as compared with wooden erections and the corrugated iron eyesore.

Signs are not wanting that the Board of Education are desirous of reducing the cost of school buildings, a Departmental Committee of that Board having made an inquiry into the same. No very definite steps appear to have been taken as yet, but among other forms the following were suggested as pointing to a more economical and temporary type of building which would allow of the alteration or rebuilding of schools to meet the ever-varying educational requirements without the excessive waste that at present obtains :—

Buildings of reinforced concrete or of concrete and steel. Steel-frame buildings with (*a*) thin walls of concrete, (*b*) hollow concrete slabs, (*c*) hollow or solid brick, (*d*) various patented materials of terra-cotta tile form, (*e*) hollow walls of metal lathing and cement mortar. Timber frame buildings cased with slabs.

In America several large season hotels have been built of steel frames with exterior walls of terra-cotta tile; also hospitals on the same lines. Small hospitals and sanatoria for consumptives will, no doubt, in future be built in this country on the lines mentioned in regard to school buildings. A good deal might be said with regard to future developments in steel frame and slab construction in regard to such erections as aeroplane and airship garages and workshops, etc. ; but such would lead into somewhat speculative spheres, and enough has been said to point out some of the lines on which advancement in fire-resisting slab construction may take place in the immediate future.



REINFORCED CONCRETE RAILWAY BRIDGE AT RIXTON FOR THE MANCHESTER SHIP CANAL CO.

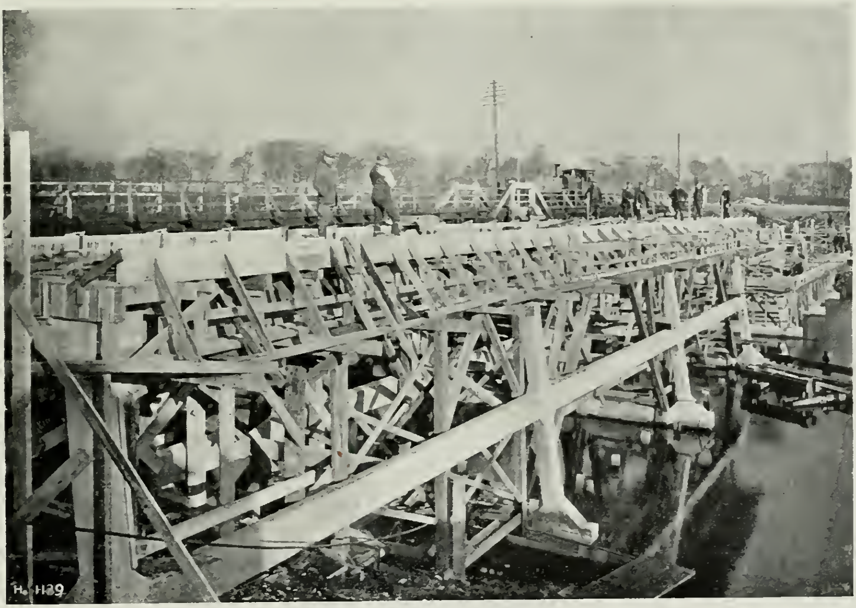
We present below some interesting particulars regarding a Reinforced Concrete Railway Bridge for the Manchester Ship Canal Co.—ED.

THE reinforced concrete bridge erected at Rixton Junction at the point where the River Mersey leaves the Manchester Ship Canal was constructed to replace an old timber bridge. It consists of eight 35-ft. spans. The main beams are supported on nine trestles. A navigation span for traffic is provided near the centre with a headway of 19 ft. 2 in. above ordinary water level.

The river bottom at the site of the bridge consists of fine sand, and, from bore holes put down at each side of the river, sand was still found at 50 ft. below the ordinary water level in the river.

Each trestle is supported on four reinforced concrete piles, 14 in. by 14 in., driven to a depth of 12 ft. below the bottom of the river.

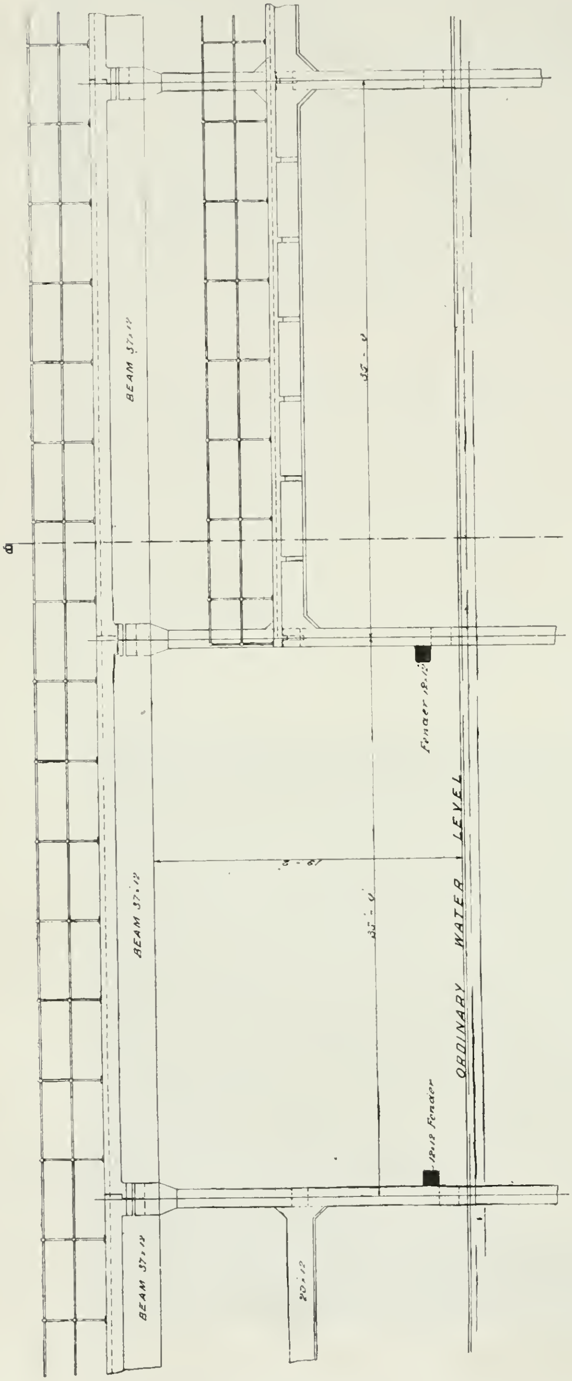
The piles were moulded with steel tubes for the application of water jets to



Bridge in course of construction.

REINFORCED CONCRETE RAILWAY BRIDGE AT RIXTON FOR THE MANCHESTER SHIP CANAL CO.

assist in driving, also for the purpose of forcing cement grout into the sand to form a base after the pile had been driven to the required depth.



Part Elevation.

Reinforced Concrete Railway Bridge at Rixton for the Manchester Ship Canal Co.

The specification provided for 15 cwt. of cement in the form of grout to be forced down each pile. This was successfully accomplished, the pressure required for the purpose being 80 pounds per sq. in.

The heads of the piles were amalgamated into concrete cross beams 18 in. by 14 in. Each cross beam is reinforced with eight 1¼-in. rods. The cross beams also form the bottom member of the A frame or trestles. The vertical members of the trestles are formed of concrete 14 in. by 14 in., reinforced with four 1$\frac{5}{16}$-in. diameter rods. The transverse members or centre ties of the trestles are formed of concrete 14 in. by 12 in., reinforced with four ⅝-in. diameter rods. The top members of the trestles are formed of concrete 14 in. by 12 in., reinforced with four ⅝-in. diameter rods. The trestles are braced together by concrete longitudinal ties 20 in. by 12 in., reinforced with four 1⅛-in. diameter rods. A gangway for bargemen is provided at the east side of the bridge. This gangway is supported on the longitudinal bracing. The gangway is formed of concrete 3 in. thick reinforced with a 4 in. by 2 in. wire mesh composed of No. 10 I.S.W.G. wires.

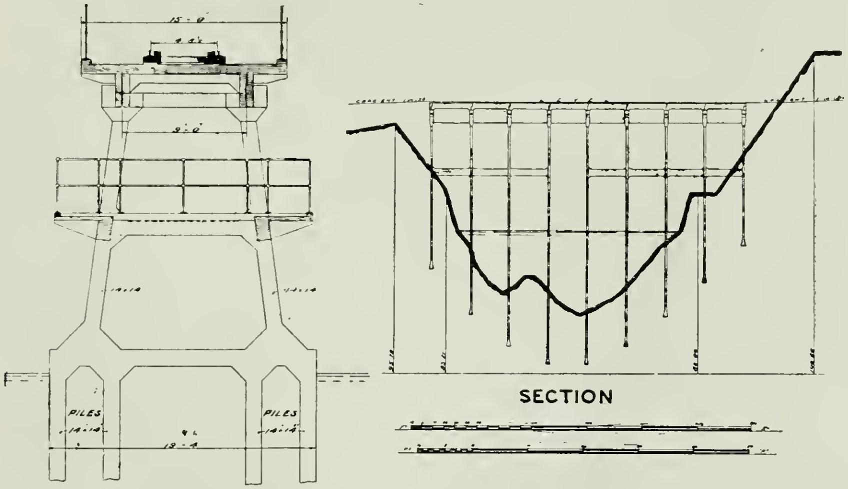
The main beams are of the ordinary type 3 ft. 1½ in. deep, reinforced at the bottom with twelve 1$\frac{5}{16}$-in. diameter rods and at the top with six 1$\frac{5}{16}$-in. diameter rods.

The bridge is a series of independent spans. Each trestle is provided with machined bearing plates embedded in the concrete at the top on which the main beams rest. The main beams are also provided with bearing plates at each end. An expansion joint is formed over each trestle. The spaces between the main beams which form the expansion joints are filled with asphalte.

The deck is a concrete slab, average thickness 8 in., cambered for drainage purposes. The slab is reinforced with $\frac{9}{16}$-in. diameter rods 4 in. pitch transversely and No. 2 I.S.W.G. wires 12 in. pitch longitudinally.

The rail baulks are secured to the concrete slab by hook bolts embedded in the concrete, and a layer of asphalte is laid over the concrete immediately under the rail baulks on which they are bedded.

The bridge is provided with hand-railing both for the gangway for bargemen and for the deck of the bridge. It is of a cheap design and effective for the purpose. The lower part of each stanchion is a steel rod $1\frac{11}{16}$-in. diameter. The upper part consists of steel cross and tee pieces. The longitudinal rails are steel pipes $1\frac{1}{4}$-in. internal diameter, which are passed through the cross and tee pieces.



SECTION AT A B

SECTION

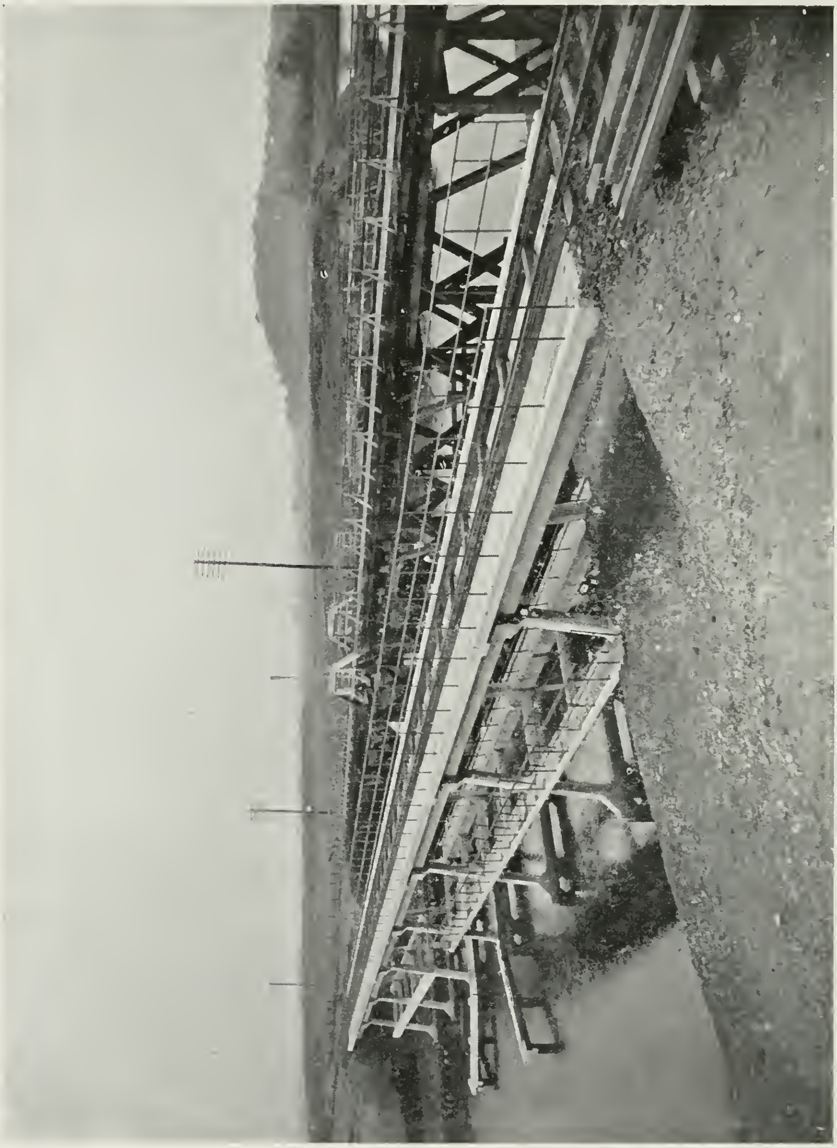
Reinforced Concrete Railway Bridge at Rixton for the Manchester Ship Canal Co.

The bridge was first tested by a locomotive with a train of railway trucks loaded with gravel. The weight of each truck was about 16 tons gross.

For the second test a load consisting of three locomotives weighing 30, 26, and 24 tons respectively, representing a load of 1·6 tons per ft. run, were run over the bridge. The locomotives were run as rapidly as possible. Deflectometers were set under each trestle supported on a stage which rested on the bed of the river and were free from the bridge. The main beams were also tested, the deflectometers being carried on the lower beams or ties of the bridge.

The permanent settlement of the trestles due to the testing of the bridge was not more than 4/10 mms. The deflection of the beams due to the locomotive and the train of railway trucks passing over did not exceed 1·4 mms. when the locomotive was passing over the instrument, and 1 mm. when the trucks were passing over. The instrument returned to zero when the loads had passed over. The deflection shown by the instrument when the three locomotives were run over was the same as that given above.

The settlement of the trestles under the above live loads was 2/10 mms. When the loads were removed the trestles returned to the original level.

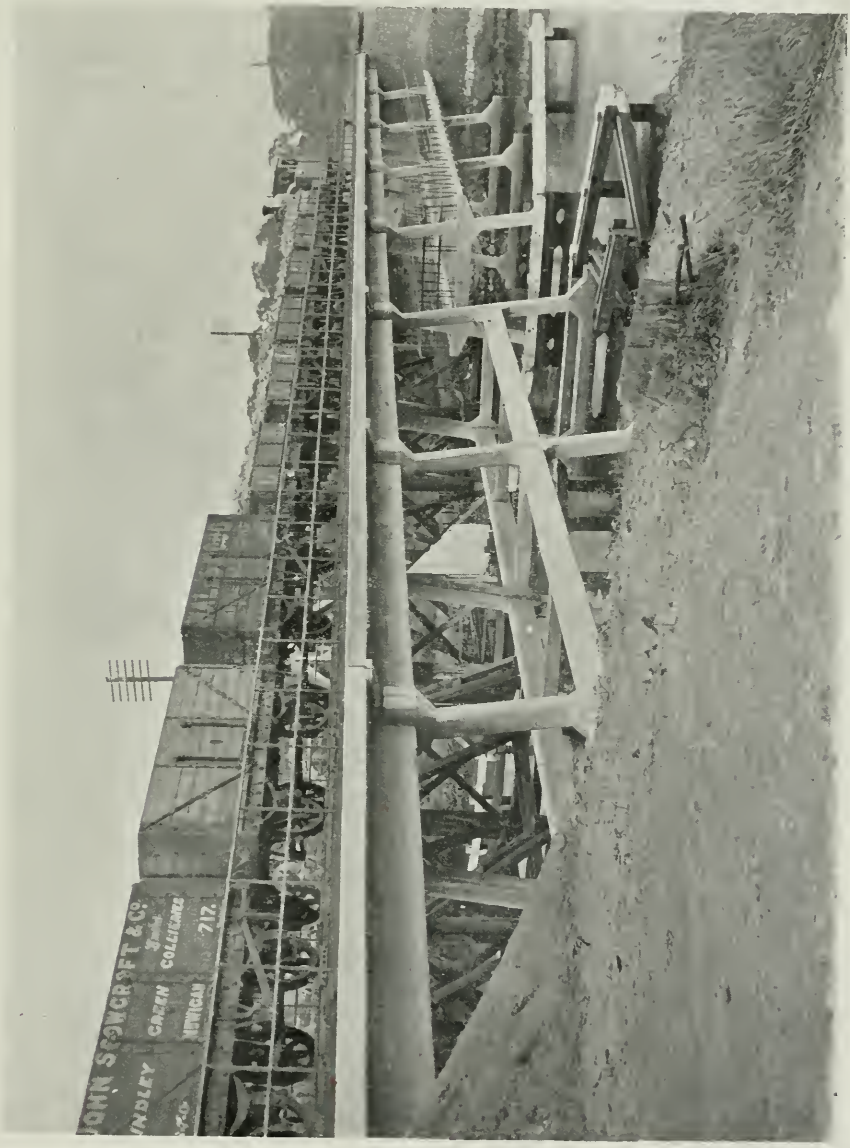


Reinforced Concrete Railway Bridge at Rixton for the Manchester Ship Canal Co.

The bridge was designed for a locomotive of 45 tons on six wheels with a 13-ft. wheel-base. The dead load was calculated as 1·14 tons per ft. run and the moving load equal to a distributed load of 2·25 tons per ft. run. The load sustained by each pile is 30 tons.

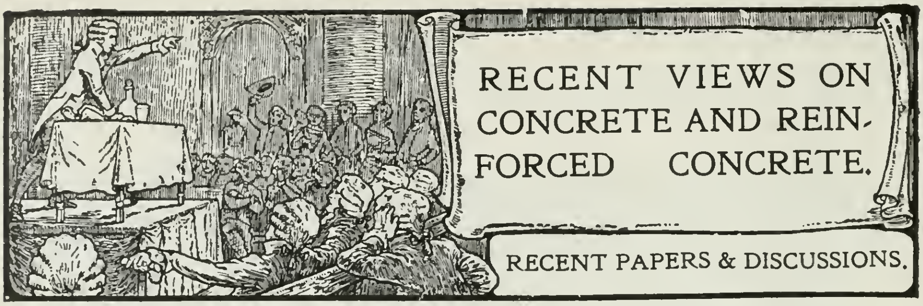
The working stresses are as follows: concrete in compression, 500 lb. per sq. in.; steel in tension, 15,000 lb. per sq. in.; and steel in compression, 15 times the stress in the surrounding concrete.

The bridge was designed by the late Mr. Hubert Congreve, M.Inst.C.E., Chief Engineer to the Manchester Ship Canal Co., and completed under the direction of the present Chief Engineer, Mr. H. A. Reed, M.Inst.C.E. The contractors for the construction of the bridge were the Liverpool Ferro Concrete Contracting Co. The Resident Engineer of this section of the canal, Mr. M. B. Lewis, had charge of the work.



View showing First Test.

Reinforced Concrete Railway Bridge at Rixton for the Manchester Ship Canal Co.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

CONCRETE IN SEWER CONSTRUCTION.

By W. J. BAIRD, B.A., Sc.

The following is an abstract of an article by Mr. W. J. Baird, B.A., Sc., written for the Canadian journal "Applied Science."

THE superior qualities of concrete as a structural material are being more fully realised every day. In almost every kind of structural work it is competing successfully with older and better known materials.

In one class of work, however, concrete has not yet reached that safe and unassailable position in the ranks of structural materials that it has in many other classes, and that is its use as a material for sewer construction. Concrete has been used for many years in the building of sewers, but cases of failure of concrete sewers have been so numerous and evidences of its reliability so conflicting, that there is even yet considerable doubt as to its adaptability under certain circumstances, and to its relative advantages compared to brick and vitrified clay pipe, the other most commonly used materials in sewers.

This condition of affairs arises from several causes; firstly, the fact that the work is covered up soon after completion often makes it difficult to discover whether any subsequent failure was due to defective material, or to an accident arising from some combination of circumstances which it was difficult to foresee and make provision for; secondly, there are few classes of work in which apparently trifling defects, such as insufficient mixing of concrete or misplaced reinforcement, may lead to such disastrous results; thirdly, when a concrete sewer fails, the fact is advertised far and wide over the country by people who are interested in the sale of other sewer materials.

Concrete sewers have been in use for many years in Europe, and to a lesser extent in America. Paris has probably more miles of concrete sewer than any other city. Brooklyn was one of the first American cities to use cement sewer pipe to any great extent; for many years their specifications required concrete pipe to the exclusion of other kinds. Wilmington, Delaware, was one of the first cities to construct monolithic concrete sewers. Many of these early sewers were constructed of natural cement concrete. Some of these have failed, and these failures have had a tendency to put concrete into disrepute as a safe and economical material. Since the introduction of Portland cement concrete into common use, however, many sewers have been constructed of Portland cement concrete, which seems to fulfil all the requirements of a perfect sewer.

In this article it is proposed to make some investigations as to how concrete sewers compare with those of other materials in—

(1) Their ability to resist the disintegrating of chemicals liable to be present in the sewage or ground water;

(2) Their ability to resist the erosive action of fluids passing through them;

(3) Their general suitability as sewage-carrying channels;

(4) Their ability to withstand the external pressures and shocks to which they are liable to be subjected;

(5) Their cost.

It is also intended to give a short description of the different forms of concrete sewers which have been constructed and some examples of sewers now in use in various cities.

Resistance to Chemical Disintegration.—The ground upon which concrete as a sewer material has been most attacked, and upon which it still remains the most open to attack, is with regard to its ability, or lack thereof, to withstand the action of chemicals with which it is liable to come into contact. It is asserted that concrete is not only liable to be disintegrated by sewage, but that it is also easily dissolved away by the action of pure water alone. The truth must lie somewhere between these two statements, just where there is none too great a wealth of evidence to show.

The disintegrating agent may arise from the sewage or from external agencies, such as the ground water. In the first case the immediate agent of destruction may be in the sewage itself in the form of dilute acids or alkalis, or it may be in the form of gases given off by the sewage. This case shall be considered first.

An often-quoted example of the disintegration of concrete by sewage gases is in connection with the sewage disposal works at Hampton, England. The history of the case was given in a paper read by Mr. Sidney H. Chambers, engineer to the Hampton Council, to the members of the Concrete Institute. In connection with the hydrolysing chamber and the inlet channel it was noticed that the concrete was being decomposed. Upon examination it was found that the concrete that was always under water was perfectly sound, while that high above the water was only slightly affected. The area of maximum decomposition was included between the low and high water levels of the sewage. The concrete where it had been attacked by the sewage gases was found to be covered with a white chalky substance which analysis proved to be sulphur. The concrete under this, upon being analysed, was found to contain a large percentage, varying from sixty-five to seventy-five, of sulphate of lime.

The explanation given for this ran as follows: the main erosive effect was at the varying liquid level, and was there dependent upon the amount of sulphuretted hydrogen in solution in the liquid. The gas was comparatively small in amount in the incoming sewage, but increased as the liquid passed through the hydrolysing chambers owing to increased putrefaction. When the level of the liquid fell it left the concrete which it previously covered, wetted with a liquid containing sulphuretted hydrogen in solution. This wet surface was then exposed to the air, which oxidised the sulphuretted hydrogen, forming sulphurous and sulphuric acids, which, acting on the concrete, changed the lime to sulphate of lime. The active agent was probably sulphurous acid, as concrete is not affected much by sulphuric acid. When the liquid rose again the decomposed material was washed away, and a fresh surface was exposed to the action of the sewage when the liquid fell again. It was the continuation of this cycle that led to the formation of the grooves at the varying liquid level.

The erosive effect on the concrete above the liquid level was dependent upon the sulphuretted hydrogen evolved from the liquid and mixed with the air supply. Some of this gas was dissolved by the moisture which was present on the walls and roof from evaporation and condensation; it was then oxidised by the air and decomposed the concrete, as described above.

The conclusions drawn from these investigations were that gases in solution in sewage and those expelled from it do not act injuriously on Portland cement concrete, provided all the following conditions occur at the same time:

(1) A high degree of putrescence in the sewage;

(2) A moistened surface which absorbs the putrid gases;

(3) The presence of a free air supply.

This case has sometimes been quoted as direct evidence of the inferiority of concrete as a sewage-resisting substance, but if a close examination of the situation is made it will be seen that there are extenuating circumstances which make the situation not as bad as it seems. In the first place, of the many sewage purification plants, this is one of the few from which there has been like trouble reported; secondly, since, as it is distinctly intimated in the report, there is necessary before sewage gases can have any effect on the concrete, a high state of putrefaction, such as could never exist in a properly designed and constructed sewer, and to attempt to introduce this as an argument against the use of concrete in sewers appears to be misapplied energy.

A somewhat similar case has been reported from Amherst, Ohio. Here the Ohio Quarries Co. have a small purification plant to care for the sewage of about eleven dwellings. This plant was built in 1905 and consists of a covered septic tank and continuous filter. Being so small there is quite a variation between the low and high levels of the sewage. Disintegration of the walls of the septic tank was noted in March, 1910, and was found to be most pronounced along the strip between the ordinary flow level and the high level caused by increased flow in times of storm. The writer gives it as his opinion that sewage will attack concrete if there is much putrefaction taking place, especially if the concrete is of anything but a dense impervious mixture.

In the reconstruction of a large sewer (15 ft. by 20 ft.) in St. Louis about eight years ago concrete was used both with and without a covering of vitrified bricks in adjoining portions of the sewer. The section covered by the vitrified brick seems to be the most satisfactory. The surface is hard and smooth and no appreciable wear can be detected. The surface of the concrete section not covered with vitrified bricks is completely pitted with small holes from one quarter of an inch to one half an inch in diameter and depth. The Portland cement mortar seemed to be partially disintegrated. When new, the mortar was allowed to harden for several days before sewage was allowed to pass over it. The cause of this pitting has not yet been determined.

In the proceedings of the Institute of Civil Engineers for November, 1908, Mr. Barnett described some investigations he made in connection with the detection of water losses in the Thirlmere Aqueduct, which supplies Manchester with water. The aggregate of the concrete was here composed of pure limestone, and it was found that where the limestone was exposed to the water, marked cuppings were visible, showing that the limestone was being dissolved by the water while the water was unaffected. Mr. Barnett made some tests to confirm this. He submitted a block of each of the following: limestone, cement mortar and neat cement, to the action of the water. He found that the limestone was the only one of the three to be appreciably affected, and it was found to have lost a considerable percentage of its weight.

It is a well-known fact that water unites with carbon dioxide gas, forming an acid which reacts chemically with limestone. Hence, water which contains a quantity of this acid will have a destructive effect on limestone with which it is brought into contact. Since carbon dioxide is evolved wherever there is any decomposing organic matter it is practically certain to be present in sewage. It would appear to be a necessary precaution to exclude limestone from the concrete to be used in sewer construction. In the results of the experiments just mentioned may lie the explanation of the action of the St. Louis sewer. However, a more detailed knowledge of all the contributory circumstances would be necessary before any definite conclusion could be made.

It would appear that fear of the concrete being attacked by gases present in the sewage is ill founded, while before it could be decided as to the probability of the sewer being destroyed by chemicals in the sewage itself, an examination of the sewage to be carried and of the concrete aggregate would have to be made. However, it is evident that any sewage having a deleterious effect on a concrete sewer would have the same effect on the cement mortar of a brick, and to a lesser extent on the joints of a vitrified pipe sewer. That brick sewers are not altogether immune from the attack of strongly acid sewage is shown in the case of a sewer in Halton Borough, England, where a brick sewer was completely disintegrated by acids allowed to drain into it from silversmiths' and goldsmiths' workshops.

The opinion of municipal engineers as to the effect of sewage on concrete is far from unanimous.

A common cause of the disintegration and consequent failure of concrete sewers is the presence of the sulphates, chlorides and carbonates of magnesium, calcium and potassium in the ground water. Of these the sulphates appear to be the most active. Cases of this nature have been noticed in the arid regions of the Western States and India, and in Prussia.

Mr. J. G. Jewett, of the U.S. Reclamation Service, gave before the American Society for Testing Materials, in 1908, certain information regarding the effect of these alkaline-charged waters on concrete culverts in Montana and Wyoming. Considerable disintegration was found to take place near the water line. This seemed to consist largely of the formation of crystals, which was accompanied by considerable expansive force. Experiments made by Mr. W. P. Headden, of the Fort Collins Experimental Station, indicate that cement concrete is very susceptible to the action of weak solutions of the above-mentioned alkali.

Preliminary to the construction of a reinforced concrete pipe line in Idaho through alkali soil some investigations were made to determine the probable effect of the alkalis on the concrete. A test length of pipe was placed in a slough where it would be exposed to the action of alkali under the most severe conditions that could be devised. Up to the present, six months later, no effect that can be detected has developed.

At Lake Loveland in 1894 a tunnel for conveying water was lined with concrete. Ground impregnated with alkalis was met with in large quantities, but by providing large and efficient side drains it was possible to keep the ground water away from the concrete, with the result that fifteen years later the concrete is apparently as good as new.

In connection with some reclamation work in the United States it was discovered that the only convenient sand supply contained 3 per cent. of alkalis, which had a deteriorating effect on the concrete in which it was used. Apparently in some regions it would be necessary to investigate the composition of sands intended to be used in making concrete.

In connection with the various failures it has been noticed that the more porous a concrete is the more liable it is to disintegration by alkali waters, and so a dense mixture of concrete should always be obtained where it is liable to be attacked by alkalis.

At the annual convention of the Illinois Society of Engineers and Surveyors the Committee on Sewers gave as their opinion that there was little to fear of concrete being injured by alkali soils in arid regions, and in any case solvents would prove as injurious to the cement mortar of a brick sewer as to a concrete sewer. Upon what evidence this conclusion was based is not recorded.

From the foregoing it may be concluded that if concrete sewers are to be built in alkali regions some, or preferably all, of the following precautions must be observed:

(1) The ground in the immediate vicinity of the sewer must be so well drained that very little, if any, ground water is allowed to come into contact with the sewer;

(2) The concrete must be made as nearly impervious as possible by using dense mixtures of concrete or by the addition of some waterproofing compound;

(3) The outside of the sewer may be protected by means of some waterproof coating, such as asphalt or tar paper.

As for the contention sometimes made that concrete is soluble in pure water, this seems too absurd to require any denial, in view of the fact that there are large numbers of concrete structures successfully withstanding the dissolving action of all kinds of ordinary water.

Ability to Resist Erosion.—There has been an idea prevalent for many years among the builders of sewers that concrete is inferior to sewer brick in its power to resist the erosive action of sediment-carrying sewage.

It is difficult to understand why concrete sewers should be less able to resist erosion than those constructed of other materials. In other structures—sidewalks, for instance—concrete shows an abrasive resistance equalled by few, if any, other materials. Experiments to determine the relative powers of resistance to abrasion of the various materials used in sewer inverts are not numerous. One series of experiments have, however, been made by Mr. G. S. Rankin, and presented in a paper read before the American Society of Municipal Improvements.

In introducing the subject of the wear of sewer inverts Mr. Rankin instances the case of a brick sewer, 6,000 ft. long, which came under his observation in which the invert was completely worn through and had to be renewed at a considerable expense. He also quotes the following instances from Folwell's "Sewerage."

"A 5½-ft. two-ring brick sewer in Baltimore, twenty-five years old, was recently found with its invert cut completely through for a width of 12 to 15 in. and badly worn to the height of 2 ft. In Omaha, brick sewers, the wear of which is usually 18 in. to 24 in. wide, became from 2 to 5 in. deep in twelve years."

In Newark, N.J., a brick sewer built in 1886, size 45 in. by 69 in., with a grade of 1·6 per cent, was found to have its invert almost completely worn through in 1910. As a result of this wearing it has been the custom to line the lower third or quarter of the cross-section with vitrified paving brick. The only test required was that the brick should not absorb more than 2 per cent. of its weight in water after being thoroughly dried and immersed in water for twenty-four hours.

Experiments were made by Mr. Rankin with the idea of determining the relative wearing qualities of several materials and their consequent suitability as materials for sewer inverts. The samples were first tested for absorption and then placed on a rubbing machine such as is used in stone yards for rubbing blocks of stone. The samples were all 20 lb. in weight and were left on the machine twenty minutes, the velocity of the rubbing machine being at the rate of 21 ft. per second. The results of the experiment were as shown in the following table:

TABLE 1.

No.	MATERIAL.	Per cent Gained by Absorp.	Per cent. Lost by Abras.	Ratio Absorp. to Abras.	Life of Invert in Years.	Remarks.
1	Vitrified Shale Brick ...	1·36	2·00	1 : 1·47	324	The life of the invert in years is calculated by assuming the life of No. 8 as 10 years.
2	Vitrified Shale Brick ...	2·15	3·35	1 : 1·56	193	
3	Vitrified Shale Brick ...	3·60	3·42	1 : 0·95	189	
4	Vitrified Shale Brick ...	4·28	5·92	1 : 1·38	109	
5	Shale Sewer Brick ...	13·18	27·31	1 : 2·07	24	
6	Building Brick	7·39	20·41	1 : 2·76	31	
7	Building Brick	10·19	44·74	1 : 4·39	15	
8	Building Brick	18·91	64·74	1 : 3·42	10	
9	1 : 2 Cement Mortar ...	11·86	4·41	1 : 0·37	147	
10	1 : 2 : 4 Grav. Concrete	6·65	3·82	1 : 0·57	169	
11	1 : 2 : 4 Grav. Concrete	5·10	3·35	1 : 0·66	193	
12	1 : 2 : 5 Stone Concrete	7·40	8·62	1 : 1·16	75	
13	Vitrified Tile	1·34	4·47	1 : 3·34	145	
14	Vitrified Pipe	4·47	6·11	1 : 1·37	109	
15	Vitrified Pipe	1·72	4·55	1 : 2·65	142	

Specimens Nos. 1—4 were vitrified shale paving brick selected for their variation in absorption. No. 5 was a small shale brick made for use in sewers. Nos. 6, 7, and 8 were different makes of building brick, also selected for their variation in absorption. Nos. 9, 10, and 12 were made from the mortar and concrete, being used in the construction of a sewer at the time of the experiment. No. 11 was a section of concrete pipe furnished by the Lock Joint Pipe Co., and was about one year old. No. 13 was a tile used by the same company for lining their pipes with, when required by engineers. Nos. 14 and 15 were samples of salt-glazed vitrified pipe.

While admitting that these experiments are far from being complete and exhaustive, Mr. Rankin draws the following conclusions from them:

(1) "That for similar materials, although there is considerable variation in the ratio, the abrasion in every case except No. 12 increases with the absorption. That gives a fair idea of the wearing qualities of the materials.

(2) "The additional cost of lining a brick sewer is warranted, although the 2 per cent. requirement of the Newark specifications seems unnecessarily severe.

(3) "The concrete sample compares favourably with paving brick and it would appear unnecessary to line a concrete sewer.

(4) "Tile pipe is generally not as durable as concrete pipe."

The last two clauses are particularly noteworthy in connection with the subject under discussion, and are somewhat contrary to the belief commonly held by many people who cannot be accused of being prejudiced against concrete in any way.

Mr. W. C. Parmeley considers that a more resisting surface can be got from concrete than from any but the hardest vitrified brick.

In Duluth, Minn., the grades of the sewers are exceptionally steep, and it has been found impossible to keep sand and small stones out of the sewers. Concrete sewers have been proven to be more durable than brick. In one case on a very steep grade, granite flags were laid in cement mortar. Two years later the granite was found to be worn away, leaving ridges of mortar between the flags, showing the concrete to be more durable than granite.

In the construction of the intercepting sewers in Toronto, brick was used in part, and brick-lined concrete in the rest. The use of brick in this case seems to have been more due to the pressure brought on the City Council by the brick interests than from any belief in the superiority of brick as a material for sewers.

In view of the extra cost involved in the use of vitrified bricks and their doubtful efficacy, the use of vitrified brick lining for sewers appears to be doubtful economy. It is always advisable to examine the grade of a sewer and the consequent maximum velocity of the sewage before it is decided whether to line it or not. In case the maximum velocity exceeds about 8 to 10 ft. per second, it would seem to be good practice to line the sewer.

General Suitability.—There are two qualities which any material must possess before it can be successfully applied to sewer construction that have not yet been taken up in detail. These are water-tightness and smoothness of surface.

Concrete is eminently fitted for sewers in that it is capable of being constructed with a very smooth surface, and all special construction necessitating the use of sharp angles and curves can be made perfectly true to shape, thus offering little resistance to the flow of sewage. This is of great advantage where there are many sharp turns in the sewer and the grade is slight.

In order that a sewer may fulfil the duties for which it was built, it must have such a form and be laid to such a grade as to cause a sufficient velocity in the water to carry along any solid materials that may be in the sewage. Upon reference to literature on hydraulics we find that a velocity of water in a pipe from $2\frac{1}{2}$ to 3 ft. per second is necessary in order to cause such materials as sand, gravel, etc., to be carried along.

The formula most used in computing the velocity of water is Chezy's.

$V = C\sqrt{RS}$ where

V = velocity in feet per second.

R = hydraulic radius.

S = size of slope angle.

C = constant depending on the slope, shape of cross-section, and roughness of surface. C is usually determined from Kutter's formula:—

$$C = \frac{41{\cdot}66 + \frac{1{\cdot}811}{n} + \frac{{\cdot}00281}{s}}{1 + \left(41{\cdot}66 + \frac{{\cdot}00281}{s}\right)\frac{n}{\sqrt{R}}}$$

where "n" is a coefficient of roughness depending on the roughness of the surface with which the water is in contact, the other symbols having the same significance as in Chezy's formula. Before proceeding any farther it is necessary to fix on the value of "n" to be used in the above formula. Merriman gives the following values for "n":

$n = {\cdot}011$ for cement mortar.

$n = {\cdot}013$ for good brick work.

$n = {\cdot}015$ for unclean surfaces in sewers.

In Ogden's "Sewer Design" the following values are given:

$n = {\cdot}011$ for cement mortar and cement pipes well jointed and in good condition.

$n = {\cdot}012$ for tough cement mortar.

$n = {\cdot}013$ for well-laid brickwork.

In the design of the concrete sewers at Louisville, Ky., a coefficient of $n = {\cdot}013$ was used. It would appear that for concrete sewers in fair condition a coefficient of $n = {\cdot}012$ would be reasonable, while in brickwork a value of $n = {\cdot}015$ would be all that could be expected. This difference in the values of the coefficients will have the effect of producing different results in determining the minimum grades in order to produce the minimum velocity in the two different types of sewers.

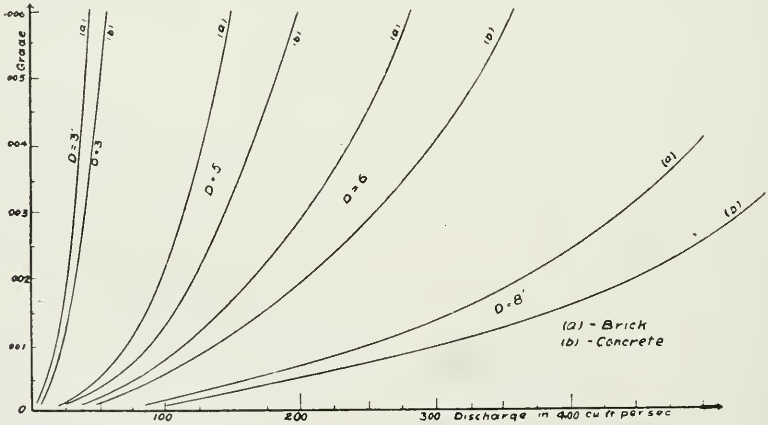
The minimum grades for pipes of various diameters consistent with the aforementioned assumptions are given in the table below :

DIAMETER OF PIPE IN FEET.	MINIMUM GRADE.	
	CONCRETE $n = \cdot 012$	BRICK $n = \cdot 015$.
2	·00150	·00260
3	·00085	·00140
4	·00055	·00092
5	·00042	·00065
6	·00034	·00053
7	·00027	·00043
8	·00022	·00037
10	·00017	·000275

Of course these figures hold true for circular sewers flowing either half or full, but the principle involved is the same whatever be the shape of the cross-section and the depth of water flowing through the sewer. That is, that concrete sewers can be laid on flatter grades than bricks for the same minimum velocity.

This table shows that in a case where the outlet has to be kept as high as possible while preserving a minimum grade in the sewer, a considerable saving in excavation might be effected by using a concrete sewer in preference to a brick one. Thus, for a 5-foot sewer the saving in grade would be ·023 ft. per hundred feet. This in a mile of sewer would mean a saving in excavation at the end of the mile of 1·12 ft. The saving in the cost of a mile of excavation in rock would be (taking the cost of rock excavation as being $3.00 per cu. yd) approximately $1,900. This in a sewer several miles long would effect a considerable saving. For earth excavation, however, the saving would probably be not more than one-sixth of this amount. Also in low-level sewers where the sewage has been pumped a reduction of the grade would mean a reduction in the height which the sewage has to be raised.

The difference in roughness of the interior surface of the two different types of sewer results also in a difference in their carrying capacity. This is shown very clearly by the accompanying curves. These show that for any particular diameter of pipe and grade the concrete sewer has a much greater carrying capacity than the brick sewer. Although the cases here illustrated are very limited in their application, the sewers being supposed of circular cross section and to be flowing full, the fact remains that



concrete sewers have a greater carrying capacity than brick sewers of the same size whatever be the shape of the cross section, or the condition of flow in the sewer. Some may not agree that there is as great difference as is found by using values of $n = \cdot 012$ and $n = \cdot 015$ for concrete and brick sewers respectively, but it is quite apparent that a higher value would be used for brick than for concrete sewers, and the values abovementioned are about as good as can be obtained at this stage of our knowledge of the flow of water in pipes. In the foregoing computations the usual assumption has been made, that, so far as the hydraulics of the case are concerned, sewage is practically all water.

It has been frequently asserted that concrete not being an absolutely water-proof material in itself, concrete sewers would be defective in that they would be liable to admit ground water and also to allow the sewage to leak out, thus polluting the ground in the vicinity of the sewer. This is, apparently, erroneous, when one considers that there are numerous concrete pipe lines all over the country which convey water, sometimes under considerable heads, without undue leakage. This would seem to indicate that it is perfectly possible to make concrete waterproof.

A concrete sewer was constructed in Denver beneath the level of the La Platte River. An effort was made to have the sewer waterproof in order to prevent seepage of the ground water. The contractor was allowed to adopt any means his fancy suggested for waterproofing the concrete, but he had to guarantee the sewer to be waterproof a year from the time of completion. The contractor used Toxement, a patented waterproofing compound, using three pounds for each sack of cement. Reliance was placed on the concrete mixture for obtaining a uniform distribution of the compound. A satisfactory result is said to have been obtained.

Preliminary to the construction of the Louisville sewerage system the commission in charge made some tests for finding the most impervious concrete. Concrete pipes were tested for about seven hours under a pressure of about fifteen pounds per square inch. The concrete used consisted of one part of cement to two parts of Ohio River sand to four parts of Ohio River gravel. Tests were made on plain concrete and on concrete containing lime, clay and various percentages of several waterproofing compounds. The best results were obtained by the use of 4 per cent. Medusa and 4 per cent. Toxement Waterproofing Compounds, the seepage in each case being zero. With the 1 : 2 : 4 concrete the seepage was 6·30 cu. in. per sq. in. of surface.

In a part of the construction of these same sewers a fine clayey moulding sand was used for 10 per cent. of the sand 1 : 2 : 4 mix. The sand was thoroughly mixed with water and then mixed with the rest of the sand and the cement. As far as can be seen this method is satisfactory.

So far as the permeability of concrete sewers and their consequent liability to leakage is concerned there seems to be little cause for alarm, provided reasonable care is used in the construction.

Stresses Due to External Pressure.—In designing sewers it is usually customary to consider the sewer as having to sustain all the dead load of the fill over it and also all the live load that is likely to come on it when the amount of cover is less than 4 ft. When the amount of fill exceeds 4ft. the sewer is supposed to be capable of sustaining the weight of the fill and a certain percentage of the live load, this percentage varying from one hundred at 4 ft. deep to zero at about 14 ft., at which point the live load is supposed to cease to have any effect. These figures are only assumptions, although probably very reasonable ones, since it is difficult to arrive at any definite conclusions from theoretical considerations alone owing to the uncertain nature of earth pressures.

In designing sewers considerable allowance has to be made for the uncertain stresses liable to be set up by irregular settlements of the foundations, and quite large factors of safety are usually considered necessary. A very complete analysis of the subject of stresses in circular sewers and culverts subjected to earth pressure is given by Prof. A. N. Talbot in a bulletin issued by the University of Illinois. In introducing the subject the Professor says : "The stresses developed in rings and pipes subject to external pressure are, of course, dependent upon the bending moments developed. As the exact load coming on the sewer and its distribution over the surface are difficult to determine, the bending moment is in general quite uncertain. The amount of load and its distribution over the surface, and therefore the bending moment on different parts of the ring, depend upon a number of conditions, such as the nature of the earth used in filling, the method of bedding the pipe, the manner of tamping the earth at the side, the amount of lateral restraint or pressure of the earth, the conditions regarding moisture, etc." These conditions of loading may include:

(1) A concentrated load at the crown;

(2) A vertical load distributed uniformly over the horizontal section;

(3) A distributed vertical load together with a horizontal load distributed perpendicularly over the sides of the ring;

(4) An oblique load, such as may prevail in a sewer trench, a slip of earth causing pressures to be set up in the sewer that are not provided for in any of the previous cases.

Mr. Talbot made a large number of tests on plain and reinforced concrete rings and pipes, comparing the results of these tests with those derived from formulæ. An examination of his results shows the chief benefits to be derived from the reinforcing of concrete pipes would be in lessening the liability of sudden failure from shock and pressures not otherwise provided for, and in permitting the use of a thinner shell for the pipe. The reinforcement should also be taken into account in fixing on a proper factor of safety. The main lesson to be derived from his tests is, however, the advisability of providing a uniform bedding for the sewer and an evenly distributed, well-compacted filling, in order that the pressures on the sewer may be distributed and not concentrated.

So far as the small number of tests made upon the relative strength of concrete and vitrified clay pipes admits of a conclusion, the resistance of the pipes against outside compression varies to a great extent, even with pipes of the same size, and made of the same materials. However, the concrete pipes seem to give slightly higher average values than vitrified clay pipes.

Average results derived from experiments are not very conclusive, and since no sewer is stronger than its weakest length, sewer pipe of material with a fairly constant and uniform strength is preferred to one of higher average strength, but which, on account of its brittleness, is more liable to sudden failure. This is one drawback to the use of vitrified clay pipes of large sizes. Their brittleness renders them liable to breakage due to careless handling or to sudden shocks from various causes.

In the construction of sewers in Richmond, Ind., vitrified tile pipe was used for sizes up to and including 18 in. diameter, while concrete was used for sewers of from 20 in. to 54 in. in diameter. The 20-in. pipe was not reinforced, and was found to be unsatisfactory; the larger sizes were reinforced, and gave good satisfaction.

The strength of the plain concrete pipes of small diameter does not seem to be much different from that of tile pipes of the same diameter, while for the larger sizes good design and proper construction is all that is required in order to obtain a thoroughly strong and reliable concrete sewer.

Comparative Costs.—The relative costs of sewers of different materials in any particular case depends largely on circumstances peculiar to that case, such as proximity to the source of manufacture. However, there are several general principles which hold true in almost all circumstances, and are applicable to a greater or less extent under all conditions, which tend to make concrete sewers cheaper than brick ones of the same size. A brief discussion of these will now be made.

Within the past few years the price of brick has advanced about 50 per cent., while the price of cement has decreased about the same amount. The market price of broken stone and sand, while governed almost entirely by local conditions, has remained nearly constant.

About 75 per cent. of the weight of the entire masonry of brick sewers, that is, the bricks themselves, must be transported from the factory to the site of the work. A large cost is usually involved in freight and teaming charges. On the other hand, in the case of a concrete sewer, frequently only about 10 per cent. of the weight, *i.e.*, the cement, has to be brought long distances, and since the quantity required is only slightly in excess of that required for brick masonry, the gain is still more apparent. The high cost of vitrified brick also tends to increase the cost of masonry structures.

A brick wall must be built of sufficient thickness to contain the line of pressure near the middle third of the ring, in order to prevent cracking. This generally results in working the materials at a very low efficiency. For example, masonry that can safely withstand a pressure of from 1,000 to 2,000 pounds per sq. in. is worked at a pressure rarely exceeding 300 pounds per sq. in., as to increase the unit pressure would produce rupture in the part of the arch subjected to tension. Concrete has the advantage that it can be reinforced with steel. With steel reinforcement the mass of the masonry may be cut down 50 per cent., and the materials comprising the structure may be worked at pre-determined and properly ascertained efficiencies.

A concrete sewer will carry a much greater volume of sewage than a brick sewer of the same size, laid on the same grade. Thus, when a given amount of sewage is to be carried, a smaller sized concrete sewer will suffice than would be required for a brick sewer. A concrete sewer 6 ft. in diameter has nearly as great a carrying capacity as a brick sewer 6 ft. 6 in. in diameter.

Supposing, in the two cases, the factors of thickness of the wall of the sewer, the cost of the material in each case to be the same, then the increased diameter will be responsible for an increase in the amount, and consequently in the cost of the material, of about 5 per cent., and for an increase in the amount of excavation necessary of about 7 per cent.

From the above theoretical considerations we would suppose that brick sewers would probably be more expensive than concrete ones of the same carrying capacity.

Many cities have discontinued the use of brick in sewers on account of the heavy cost. In the construction of the high-level intercepting sewer in Toronto, certain sections of it were built of concrete lined with brick, and others of brick alone. In the sections where concrete was used the economy was very marked.

The cost of concrete pipes varies greatly under differing conditions, but in general they are cheaper than vitrified tile pipes of the same size.

In general, concrete sewers seem to be cheaper than those of other materials, in many cases by a considerable amount, and very seldom, if ever, are they more expensive than those of brick or vitrified tile construction.

CEMENT PAVING AS CONSTRUCTED AT MASON CITY, IOWA.

By MR. F. P. WILSON, City Engineer.

The following is an Abstract of a paper read before the National Association of Cement Users at their meeting in March last, by Mr. F. P. Wilson.

I HAVE endeavoured to describe in this paper my experience and the methods I have used to construct first-class Portland cement concrete pavements in Mason City, Iowa.

In the past five or six years I have made a broad study of cement paving, and have visited a large number of cities in the Northern States where this class of pavement has been laid, and I watched the methods of construction in detail.

It is my opinion that a Portland cement concrete pavement, properly laid in an up-to-date manner with first-class cement, good, clean, sharp sand, and good, clean, hard stone, making proper allowance for expansion and contraction, certainly warrants its use on account of its first cost, low cost of maintenance, its cleanliness, and the small expense to repair when it becomes necessary to cut holes.

In constructing a thoroughly good cement pavement, the primary requirement is first-class material; secondly, an up-to-date set of plans and specifications; and lastly, a rigid and close following of these specifications in every detail.

The following specification has been used with entire success by me at Mason City:—

Preparation of Roadbed.—All streets, prior to the laying of any pavement thereon, shall be graded so that the pavements may be at the established grade when completed. After excavating the sub-grade, unless the engineer deem the natural ground a proper foundation, excavation shall be continued until solid ground is reached, and then refilled to sub-grade with sand, gravel and broken stone.

The contractor shall be required to remove at his own expense all obstructions, such as trees, old blocks, débris, etc.

Excavation.—All excavated material, gutter stones, planks, macadam, crossing-stones, old curbs, surplus earth, etc., shall be the property of the city, and be deposited by the contractor in such place and manner as shall be directed by the engineer. the distance not to exceed 3,000 ft. No ploughing will be allowed within 3 in. of the bottom of the foundation.

Rolling.—When the street has been graded and shaped to its proper form, it shall be thoroughly rolled with a 10-ton roller to a thoroughly compact surface. If the ground is wet, sand or gravel is to be put in before rolling.

Any depression discovered after this rolling shall be refilled to sub-grade, re-rolled, and this repeated until a roadbed, perfect as to grade and form, shall be made.

Tamping.—When the use of the roller is impracticable, the foundation must be thoroughly puddled and rammed until compacted to the satisfaction of the engineer.

Concrete Foundation.—Upon the roadway thus formed will be laid Portland cement concrete 5 in. thick, to be made as follows: one part by measure of Portland cement; two parts by measure of clean, sharp sand, and five parts by measure of broken stone.

The sand and cement shall be thoroughly mixed into mortar at the proper consistency with a batch mixer approved by the engineer. Broken stone, thoroughly cleaned of dirt, drenched with water, but containing no loose water in the heap, shall then be added to the mortar in the proper proportion. The concrete will then be turned and mixed until each fragment is thoroughly coated with mortar, a strictly wet mixture being required. The concrete thus mixed shall have such a consistency that when rammed the mass will not shake like jelly, but will, when struck, compact within the area of the face of the hammer without displacing the material laterally.

The concrete thus prepared shall be placed immediately in the work. It shall be spread and thoroughly compacted until free water appears on the surface, which shall be made smooth and parallel to the surface of the finished pavement. The whole operation of mixing and laying of each batch of concrete shall be performed in an expeditious and workmanlike manner, and be entirely completed before the cement has begun to set.

No retempering of concrete will be permitted, and concrete in which the mortar has begun to set will be rejected.

The thickness of this concrete to be 5 in. after the same has been compacted.

Extreme care should be taken that the sub-grade is kept moist while this concrete is being put in place.

No concrete shall be laid when the temperature at any time during the day or night falls below 35° above zero Fahrenheit.

Wearing Surface.—Upon the concrete heretofore specified shall be immediately laid a wearing surface 2 in. in thickness, to be made as follows: one part by measure of Portland cement; two parts by measure of coarse, clean, sharp sand; the sand and cement shall be thoroughly mixed into mortar of the proper consistency with a batch mixer approved by the engineer.

The mortar thus mixed will be immediately laid upon the concrete heretofore specified.

Before this mortar has begun to set it will be finished off to a smooth surface with a wood float; and before completely hardened it shall be roughened by brushing with a stiff vegetable brush or broom.

The curvature and cross-sections of the pavement to be made according to the plans governing the same.

Requirements of Materials.—The cement used in the work will be submitted to the tests approved and recommended by the American Society of Civil Engineers, which it must stand to the satisfaction of the engineer.

All Portland cement used in the work shall be Mason City Portland cement, or other Portland cement equally as good, which shall be protected from the weather, free from exposure to air slacking, and from moisture.

The sand shall be clean, sharp sand.

The stone used for the concrete shall be of the best quality of hard lime stone, or other stone equally as good, and shall be broken to such a size that the fragments shall not be larger than will pass through a 1 and 1½-in. ring, and not smaller than a hazel nut. It shall be free from dust, dirt, loam, or other objectionable material, and shall be screened, when necessary, over ½ in. screen to eliminate dust and small particles.

Expansion Joints.—An expansion joint 1 in. in width shall be left next to the curb on each side of the street or alley, also an expansion joint ½ in. in width will be left every 25 ft. across said pavement at right angles to the curbs. Said expansion joints are to be filled with an asphalt-paving filler of proper quality and consistency approved by the engineer. It will be applied while heated to a temperature of about 400° Fahrenheit, and shall be so applied that said expansion joints shall be thoroughly filled clear to the top of the surface of said pavement.

All forms for expansion and contraction joints shall be made of iron or steel in the form of a template, cut to the desired shape of the street, according to the plans, and of sufficient strength to resist springing out of shape. All mortar and dirt shall be removed from forms that have been previously used. The forms shall be well staked to the established lines and grades.

Contraction Joints.—Contraction joints shall be made entirely through the pavement every 12½ ft. at right angles with the street.

The edges of all expansion and contraction joints shall be rounded to a radius of about ½ in. with proper tools.

Care should be taken to obtain a surface free from ridges, at expansion or contraction joints, and depressions or unevenness in the surface, that will detract from its appearance, or cause water to lay on the pavement.

Any section having such inferior surface will be rejected, and shall be rebuilt by the contractor at his own expense.

Care shall be taken to make the expansion joints in such a manner that they are practically the same width throughout their depth.

Extreme care must be exercised in removing templates or divisions used to make expansion or contraction joints; the breaking out of any portion of the pavement in removing such templates and forms will not be tolerated, and such damaged portions of the work shall be torn out and replaced in good condition by the contractor at his expense.

The contractor shall keep the pavement sprinkled for one week after it is laid, or longer if deemed necessary by the engineer.

The contractor shall keep the streets barricaded where pavement has been laid at least two weeks after the completion of the same.

In the construction of this work a mechanical batch mixer with a 25 ft. boom, with a traveller, was used. After the sub-grade had been thoroughly rolled, the material was distributed along the street, the rock on one side and the sand on the other. The mechanical mixer was set up at the end of the street. The concrete was placed in the first section. The wearing surface was immediately placed upon the concrete, and not more than twenty minutes were allowed to elapse between the time the concrete was placed and the wearing surface was put on the same. The next section was dealt with in the same way. Then the mixer propelled itself backwards 25 ft. and proceeded as before.

Parallel with the curb, and 10 ft. out from the same, the wearing surface was cut through into the concrete, the parallel cuts being 10 ft. apart, so that our actual blocks of concrete are only 12½ ft. by 10 ft.

Some of the streets where the cement paving was laid were very soft and swampy. To obtain a dry and well-drained sub-grade a trench parallel with the curb on each side of the street was excavated, and a 4-in. drain tile was laid in same connected with the sewers. The earth excavated from these trenches was hauled away, and the trenches were refilled with good, clean, hard burnt cinders, making a thorough drainage for the sub-grade.

The contract price for these cement pavements, including excavations, was $1.32 per sq. yard.

I am changing my specifications for future work by demanding that my expansion joints be placed 37½ ft. apart at right angles to the curb, instead of 25 ft. apart. I am also requiring a softened steel reinforcement plate ¼ in. in thickness and 2 in. in depth, to be anchored back into the wearing surface; these steel plates to be used on each side of the expansion joint to protect the edges of the pavement at that point.

The reason for using these expansion protection plates is that the weakest point of a cement pavement is at the expansion joints.

NEW WORKS IN CONCRETE
AT HOME AND ABROAD.

Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

CONCRETE BLOCK COTTAGES, KILKENNY.

In our last issue we published as our frontispiece a photograph of six interesting concrete block cottages at Talbot's Inch, Kilkenny.

We have now obtained detailed plans showing the ground and first floors, and these being of some importance, we reproduce these drawings on the following page.

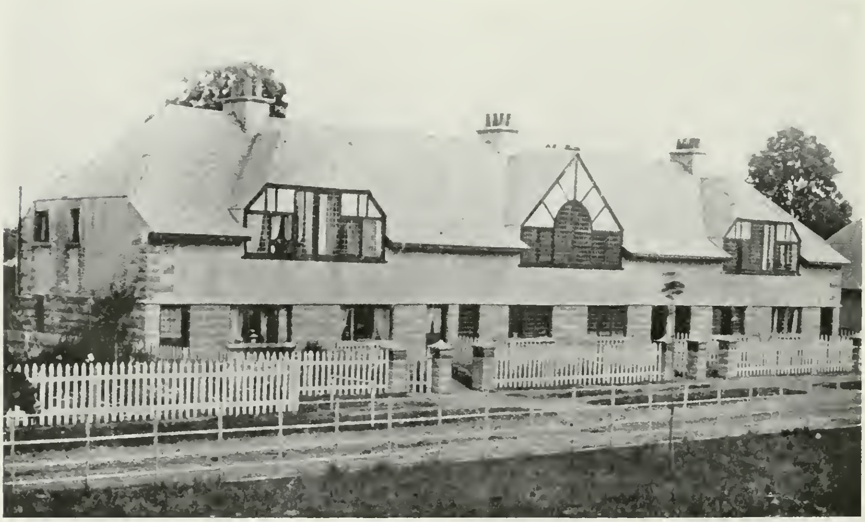
The cottages have been erected in connection with a further extension of the Kilkenny woodworkers, woollen mills, and tobacco plantation industries which are being fostered under the patronage of the Rt. Hon. Ellen Countess Dowager of Desart, for whom they have been designed and erected by Mr. F. W. Kiddie, architect, of Talbot's Inch. The plans show a most economical arrangement of rooms, no space being lost, the provision of a large living room and good sized scullery, containing a bath, being particularly suitable for the cottager. It will be noted that three bedrooms are provided. The grouping of the fireplaces is a most economical arrangement, as one set of breasts and stacks serves two houses.

The outside appearance is most artistic and pleasing, the contrast afforded by the "Rock"-faced blocks on the ground floor and rough cast on the first floor, and again by the red fibro cement tiles, being extremely satisfactory.

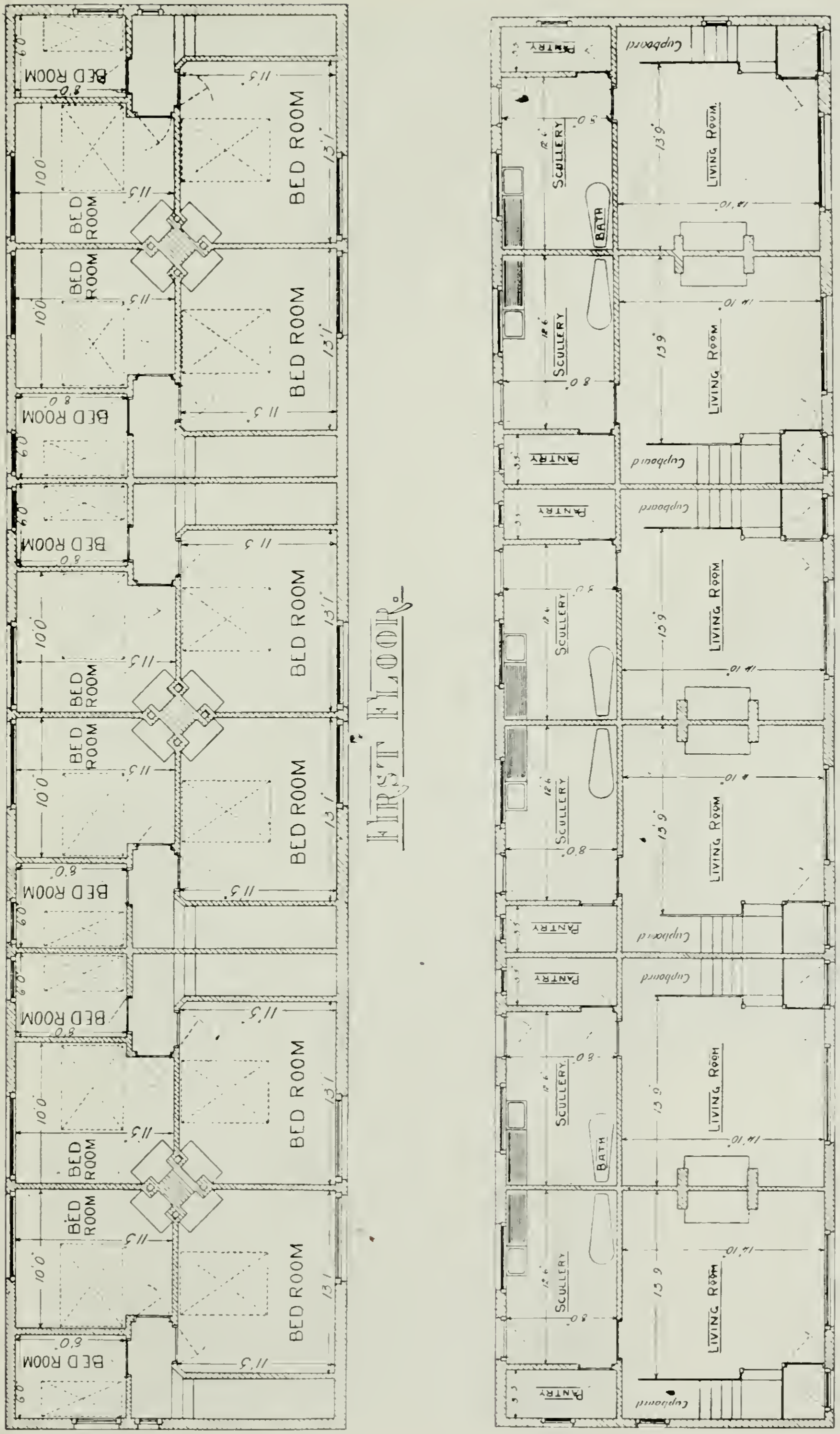
The large gabled windows, which really serve two cottages, are a feature which quite takes the houses out of the commonplace of cottage elevations.

The white-paled fence gives a clean and neat finish to the whole, while the "Winget" block gate piers add to the substantial appearance.

The whole scheme is a marked indication as to the way Portland cement is entering into modern life. The outer walls are built with "Winget" cement blocks, the



Concrete Block Cottages at Talbot's Inch, Kilkenny.



First Floor.

Ground Floor.

Plans of Ground Floor and First Floor.

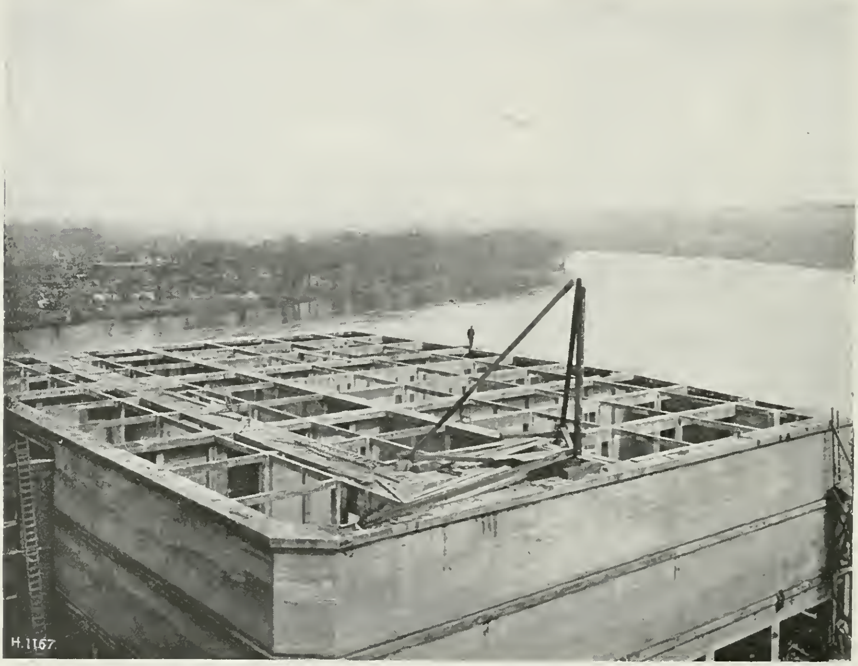
Concrete Block Cottages at Talbot's Inch, Kilkenny.

partition walls with "Winget" cement slabs, the floors of concrete, and the roof with fibro cement sheeting and fibro cement tiles.

By the bye, in our previous issue the frontispiece illustrating these cottages bore a wrong reference. The page named should have been p. 785.

10,000 TON REINFORCED CONCRETE COAL POCKET.

The accompanying illustration shows a 10,000 ton reinforced concrete coal pocket of the Pacific Mills Print Works, located on the south bank of the Merrimac River at South Lawrence, Mass., and is probably one of the largest coal pockets ever built of concrete. It was constructed by the Aberthaw Construction Co., Boston, in accordance with the design of Lockwood, Greene and Co., of the same city. It extends 119 ft. 7 in. to the east of the boiler-house, and is 153 ft. 3 in. in length, with a partially glazed monitor running its length on the western side and over the coal conveyer delivery. The roof is of 5-ply tar and gravel on 3-in. plank, supported by 7¾-in. square pine columns, with 8 in. by 12 in. and 10 in. by 14 in. pine columns.



Reinforced Concrete Coal Pocket.

The concrete piers rest on Simplex concrete piles. In the basement there are six longitudinal rows of 37-in. diameter reinforced concrete columns spaced generally 16 ft. 9 in. on centres. These columns support the heavy buttresses of the coal-pocket floor, which slopes from each column down to the cast-iron thimbles, through which the coal is discharged to the cars running on the industrial tracks in basement below. The coal pocket proper is about equally divided lengthwise by a concrete wall, and it is divided across in thirds by two other concrete walls.

The coal is received from the railroad cars that run over a hopper located at the south-west end of the pocket. After passing through the hopper the coal goes through a Hunt coal cracker, and is automatically emptied into buckets of a Hunt noiseless conveyor system, and is then taken immediately up to the roof. The conveyor driver is operated by motor.

The C. W. Hunt Co. furnished all the coal-handling machinery according to specifications of Lockwood, Greene and Co.

Reinforced Concrete Culvert at Spital, near Chesterfield.

REINFORCED CONCRETE CULVERT AT SPITAL, NEAR CHESTERFIELD.

The accompanying illustration shows a culvert in reinforced concrete which has recently been constructed at Spital, near Chesterfield, by Messrs. Evans Bros., contractors, of Alfreton. This work has been erected on the Coignet system.

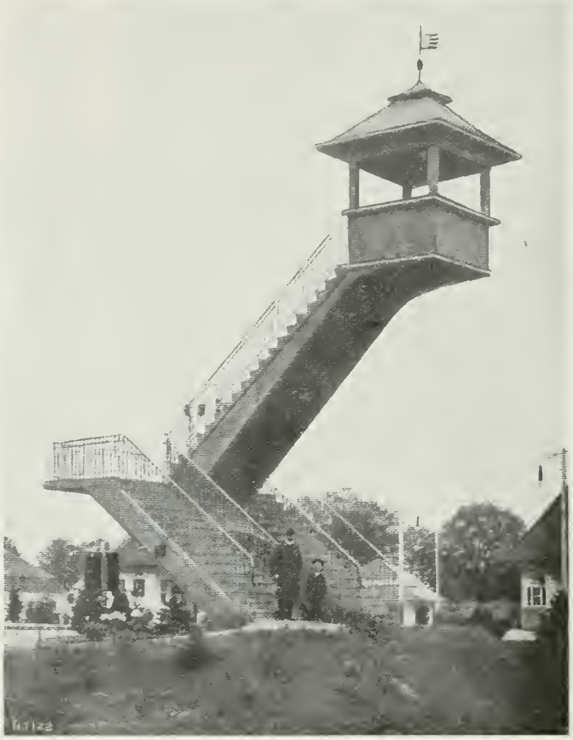
The culvert, which is 90 ft. long by 6 ft. internal diameter, was built in situ and in a continuous manner without any joints. It is built in the course of a brook, which in ordinary times is only about 1 ft. in depth, but after a storm it is anticipated that the culvert may be filled to its full capacity.

The reinforcement is composed of a meshwork of plain, round bars of mild steel. The culvert is to be filled over to a height of 15 ft. of earth, and on the top of this filling a road will be made to carry heavy traffic. This structure will join the villages of Spital and Haslend, both in the borough of Chesterfield. The work has been erected for Alderman C. P. Markham, of Chesterfield, and is being given to the town by him.

A REINFORCED CONCRETE STAIRCASE.

We present herewith an interesting illustration of a staircase constructed in reinforced concrete, erected at the Industrial Exhibition held in Köslin, Pomerania, in the summer of this year. The staircase terminates in a pavilion, which is also constructed in reinforced concrete.

This is certainly a remarkable instance of what can be done in this form of construction, and gives an excellent indication that, where steel frame is not available, equally good results can be obtained with the aid of Portland cement and ordinary bars. As to some of the dimensions, the floor of the pavilion is about 22 ft. above ground-level, and the total height to the top of the pavilion is about 32 ft., and the carrying capacity is calculated at 500 k. per sq. m.



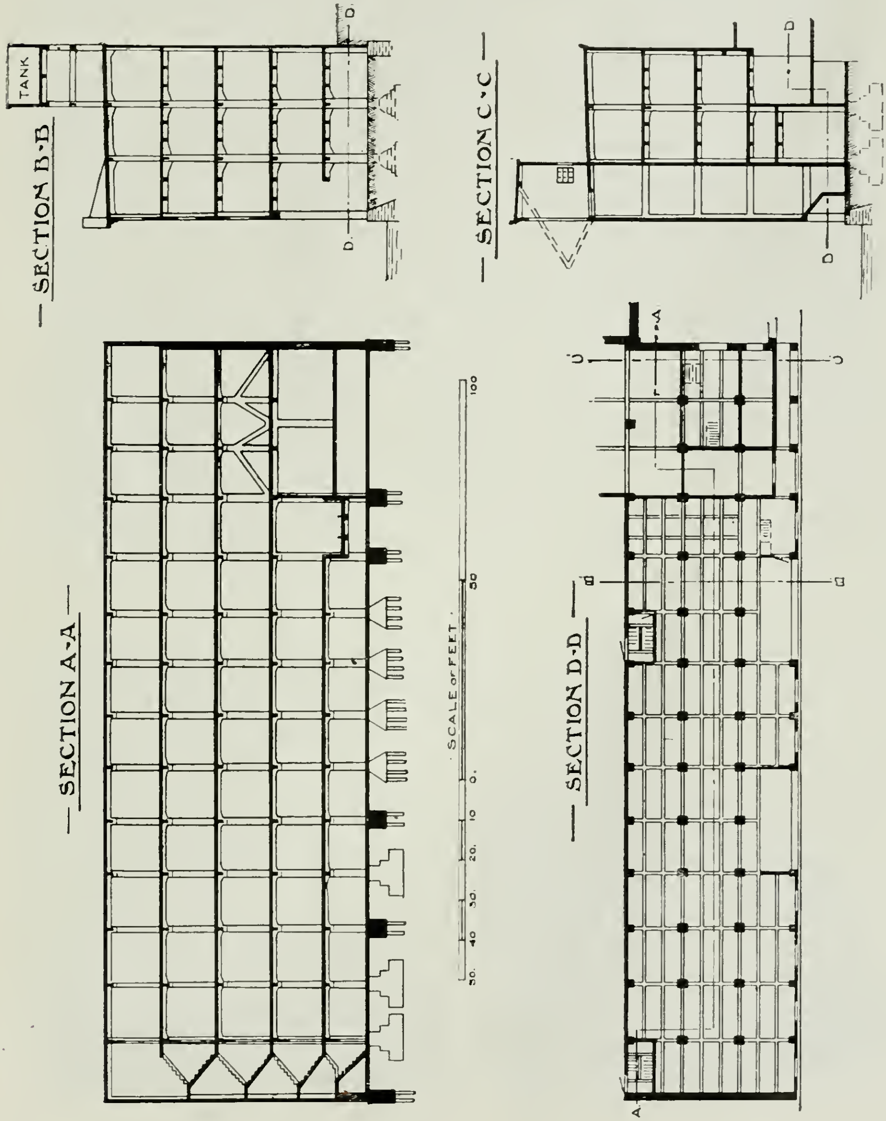
A Reinforced Concrete Staircase, Köslin, Pomerania.

The staircase was prepared from a design of Professor Köhnke, of the Technical High School, Danzig. The construction was carried out by Messrs. Wayss and Freytag, A.G., Berlin.

REINFORCED CONCRETE OIL MILLS, HARBURG (GERMANY).

The accompanying illustrations show some oil mills erected for the "Teutonia" Oil Works in Harburg-on-the-Elbe. They were constructed throughout in reinforced concrete. The length of the building is 187 ft., the depth about 42 ft. 6 in., and the height about 65 ft. 6 in., there being five storeys.

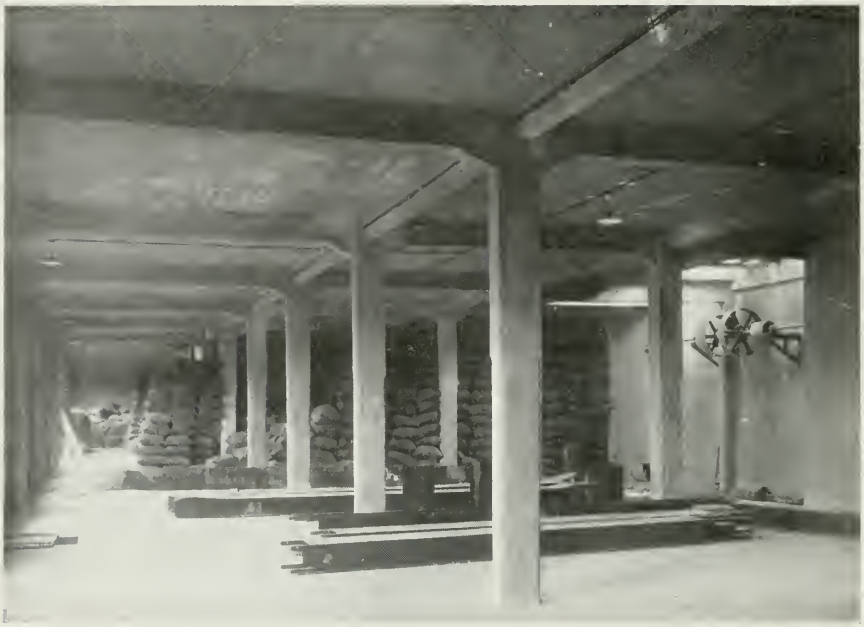
Owing to the heavy loads that have to be carried, it was necessary to construct a network of beams in both directions, which at the same time adds considerably to the stiffness of the whole building. All the walls are of a thickness of about $3\frac{3}{4}$ in., with double reinforcements. One side of the building facing the canal is furnished with cranes and hoists for taking barge loads, such as coal and raw materials, into the warehouse, while the other side has a very easy connection to the railway, a track being carried right into the ground floor. The engine-house is part of the building, and is also carried out in reinforced concrete. Adjoining the boiler-house (which is the



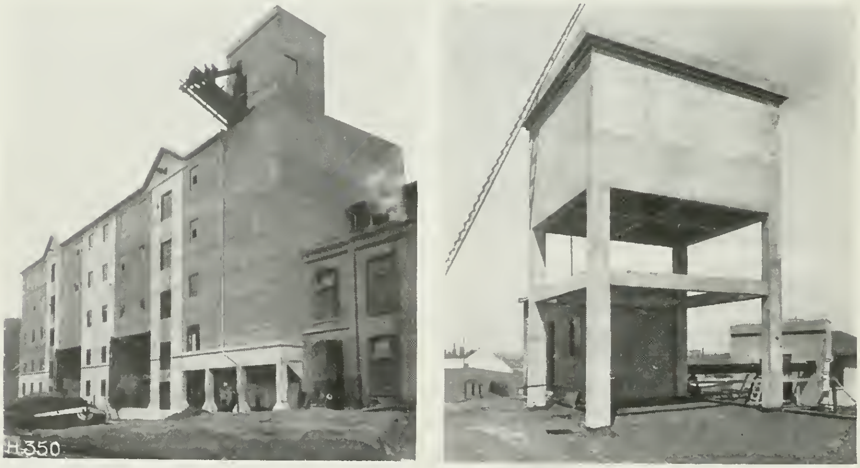
"Teutonia" Reinforced Concrete Oil Mills, Harburg (Germany).

next building) is a coal silo which has a capacity of about 19,000 cub. ft. Two flights of stairs, divided from the warehouse by reinforced concrete walls and iron doors, afford the means of communication between the different storeys. On the roof a water tank has been constructed. This tank has a capacity of 1,250 cub. ft.

These mills were erected to the designs and under the supervision of Messrs. Christiani and Nielsen, of Hamburg and Copenhagen.



Reinforced Concrete Oil Mills, Harburg (Germany).



Reinforced Concrete Oil Mills, Harburg (Germany), showing Water Reservoir on Top.

NEW BOOKS

AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"Retaining Walls for Earth: including the Theory of Earth-Pressure as developed from the Ellipse of Stress, with a Short Treatise on Foundations, Illustrated with Examples from Practice." By Malverd A. Howe, *C.E., Professor of Civil Engineering, Rose Polytechnic Institute, M.Am.Soc.C.E.*

London: Chapman & Hall, Ltd. New York: John Wiley & Sons. Price 5/6 nett, or $1.25. Fifth Edition, revised and enlarged. Small 8vo, xii + 182 pp.

Contents.—Theory of Earth Pressure—Formulas for Earth Pressure—Stability of Trapezoidal Walls—Formulas for Trapezoidal and Triangular Walls—Foundations for Walls Retaining Earth—Examples—Foundations—Foundations under Water and Deep Foundations—List of References—Tables—Appendix A: Reinforced Concrete Retaining Walls—Appendix B: Profiles of Gravity Walls Retaining Earth.

Many writers have contributed to the theory of earth pressure, and the list of references given by Prof. Howe in this little volume is formidable, but he is able to show that all theories bear a family resemblance, and are ultimately based upon the same premises. The fact that his book has reached a fifth edition proves it to be a valuable one, and for students it gives the usual or more general treatment of the problem in a "nutshell" almost. This edition has been improved beyond the former editions by re-writing and bringing up to date the notes on reinforced concrete retaining walls, which now are used to a very great extent by reason of their economy over other types. The notes on foundations are suitable for college students perhaps, but not adequate enough as a practical or indeed a thorough theoretical treatise on the subject; but after all they are merely a side issue to Prof. Howe's main subject—the theory of earth pressure. Now scientific theory is based on practice and experiment. Prof. Howe gives us examples of practice in the shape of walls that have been built, but neither he nor his authorities have done enough experimenting. There have of recent years been other theories put forward that are fundamentally different from Rankine's method of conjugate stresses, or the alternative method of the wedge or prism of maximum pressure which is the same as regards the mathematical basis. The theories we refer to are those of Mr. Meem and Mr. Haines, brought before the American Society of Civil Engineers in 1907. They attempt to account for experiences that the other theory does not explain, and however wrong they may be deserve attention. Prof. Howe would do well not to ignore such theories in any further edition, but consider the conditions to which they refer, and if possible experiment and put forward some explanation. Nevertheless, the book is an excellent elementary one on the subject and deserves every commendation.

"Transactions and Notes of the Concrete Institute." Vol. IV., Part II.

Published at the Offices of the Institute, Denison House, 296 Vauxhall Bridge Road, Westminster,

This volume contains a report of the Institute's meetings during the earlier part of this year, and includes papers presented by Prof. Beresford Pite on the Æsthetic Treatment of Concrete; Mr. Reginald Ryves, Assoc.M.Inst.C.E., on High Dams of Great Length; Mr. Maurice Béhar on the True Bending Moments of Beams. It further contains the Report of the Institute's Tests Standing Committee on the Testing of Reinforced Concrete Structures on Completion. As we have dealt with these papers at some length in former issues it is not proposed to deal with them in detail here. The volume is well illustrated.

Among the announcements in the Transactions we note that the membership comprises 878, and that there are 23 students and 9 special subscribers. We also note that there have been a number of useful additions to the library.

We would also like to draw attention to the very excellent design of the medal which was presented to Prof. A. Beresford Pite.

CORRESPONDENCE.

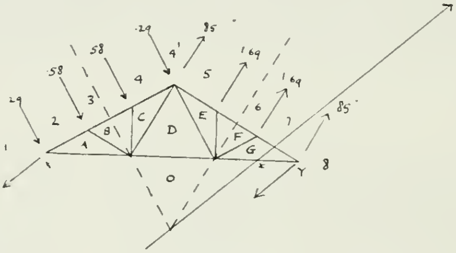
Under this heading we invite correspondence.

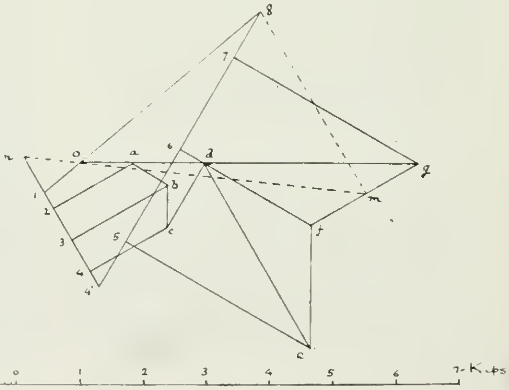
WIND PRESSURE ON ROOFS.

SIR,—The article on the above subject by Mr. Ewart S. Andrews, B.Sc., in the September number of CONCRETE AND CONSTRUCTIONAL ENGINEERING is very interesting, as giving a practical method of applying the principles disclosed by the National Physical Laboratory experiments. The following point, however, seems to have been lost sight of.

The experiments referred to determined the *intensity* of pressure and suction at the centre of the principal rafters of the roof models. Now the 1903 experiments tended to show that, while the distribution of suction over the leeward rafter is practically uniform, that of the pressure over the windward rafter is by no means so;* that the point of maximum intensity of pressure is near the centre of the rafter, or a little below it; and that the pressure vanishes at the ridge and decreases materially towards the eaves. Irminger's experiments many years before gave somewhat similar results.

The practical meaning of this seems to be that in calculating the total wind-pressure on the windward side, some value less than 15 lb. per foot super may be taken; and the present writer suggests that it will be found sufficiently accurate to consider the total windward pressure to be uniform over the rafter at *half* the maximum intensity, *i.e.*, at $7\frac{1}{2}$ lb. per foot super.





It would be complicated and unpractical to attempt to define exactly what pressure comes on each of the windward purlins, and no great accuracy of design would be obtained by doing so. All that is required is that the amount and line of action of the total windward pressure should be obtained as accurately as possible. The division of this among the joints of the windward rafter only affects the wind-stresses in the members on that side of the truss, and these stresses will be found to be so small as to have no effect on their design.

The figures show the space and reciprocal diagrams for the same truss as that worked out by Mr. Andrews, the windward pressure being taken as $7\frac{1}{2}$ by 23·09 by 10 lb., acting at the middle point of the rafter; and the table gives the results alongside of those of Mr. Andrews. It will be seen that, according to this method of working, the main tie-rod gets very little compression, and its design might not be affected thereby; while the only other members whose design will be greatly affected are the ties C D, D E, each of which must be capable of taking 1,290 lb. compression.

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* Proc. I.C.E., vol. clvi p. 105.

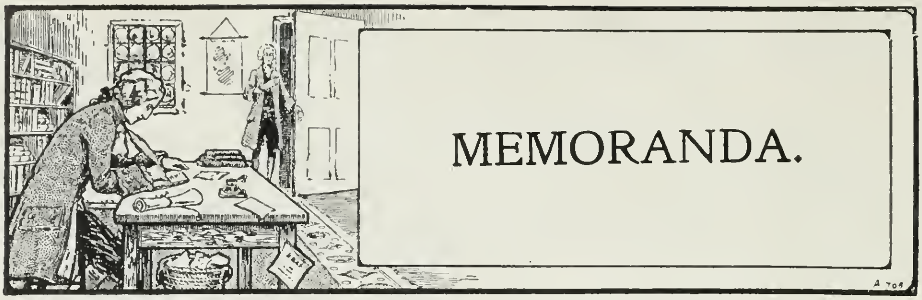
TABLE STRESSES IN THOUSANDS OF POUNDS.

+ = compression. — = tension.

Member,	Dead Load.	Wind.		Maximum Stress.		Minimum Stress.	
		Left.	Right.	As now calculated.	Mr. Andrews	As now calculated.	Mr. Andrews
A 2	+6·00	−1·43	−3·39	+6·00	+6·06	+2·61	+3·65
B 3	+4·80	−1·74	−2·40	+4·80	+4·22	+2·40	+3·40
C 4	+4·80	−1·43	−3·39	+4·80	+4·86	+1·41	+2·45
A o	−5·20	+·83	+5·35	−5·20 +·15	—5·20 +2·28	—4·37	—3·36
D o	−3·12	+1·95	+1·95	—3·12	—3·12 +1·13	—1·17	+1·13
AB	+1·20	+·66	−1·95	+1·86	+2·50	—·75	—·73
BC	+1·20	+·66	−1·95	+1·86	+2·50	—·75	—·73
CD	−2·08	−1·20	+3·37	−3·28 +1·29	−4·38 +1·25	+1·29	+1·25

It is an open question whether the possibility of the truss sliding on its expansion end—so that the reaction there is, perforce, vertical—should be considered. It would not be found, however, that any great increases in wind-stresses would be caused thereby.

C. R. SATTERTHWAITE,
Lieut., Royal Engineers,
Assistant Instructor in Construction School of Military Engineering, Chatham.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The Concrete Institute, Session 1912-13.—The Concrete Institute announce the programme of meetings for their fifth session, 1912-13, the meetings to be held in the Lecture Hall at Denison House, Vauxhall Bridge Road (close to Victoria Station), Westminster, S.W., and called for 7.30 p.m. The programme comprises the following items:—November 14th—Presidential Address by Mr. E. P. Wells, J.P. November 28th—Paper by Mr. John M. Theobald, F.S.I. (Member), on "Bills of Quantities for Reinforced Concrete Work." December 12th—Papers by Mr. Robert Cathcart (Member) and Mr. Laurence Gadd, F.I.C. (Member), on "The Effects on Concrete of Acids, Oils and Fats." January 9th—Paper by Mr. W. Valentine Ball, Barrister-at-Law, on "Concrete in its Legal Aspect." January 30th—Paper by Mr. Percival M. Fraser, A.R.I.B.A. (Member), on "Reinforced Concrete Factory Buildings." February 13th—Paper by Mr. S. Bylander (Member) on "Three Steel-Frame Structures in London." February 27th—Paper by Mr. John A. Davenport, M.Sc. (Vict.), B.Eng. (L'pool) (Member), on "Economy in the Design of Reinforced Concrete." March 13th—Discussion of Reports of the Reinforced Concrete Practice Standing Committee on: (1) Cracks in Concrete, (2) Surface Treatment of Concrete. March 27th—Paper by Prof. Stephen H. Dixon, M.Sc., M.A., M.Inst.C.E., Professor of Engineering at Birmingham University (Member), on "Beams and Props for Mines." April 10th—Paper by Mr. E. Fiander Etchells, F.Phys.Soc. (Member), on "Structural Engineering." April 24th—Discussion on Reports of the Science Standing Committee on: (1) A Standard Notation for Structural Engineering Calculations. (2) A Standard Specification for Reinforced Concrete Work. (3) Standard Connections and Joints in Reinforced Concrete. May 8th—Paper by Mr. H. Kempton Dyson (Secretary) on "Shear and problems arising therefrom." May 22nd—(4.30 p.m.), Fourth Annual General Meeting, followed at 8 p.m. by the Third Annual Dinner.

This programme is a very instructive one, and it alone should induce many professional men and others concerned to join the Institute, apart from the fact that it distributes some very interesting "Transactions." Students, it should be remembered, are also entitled to both literature and attendance at the lectures.

Educational Lectures on Concrete.—The Concrete Institute has very wisely arranged for a course of six educational lectures on "Concrete: Its Properties and Manufacture," to be given by its Secretary, Mr. H. Kempton Dyson, who acts as lecturer on reinforced concrete and structural engineering at the London County Council School of Building, and has considerable experience in technical tuition. The lectures will be given in the Lecture Hall of the Concrete Institute, at Denison House, Vauxhall Bridge Road, Westminster, on November 12th, 19th, 26th, December 3rd, 10th, and 17th, at 5.30 p.m., and the lectures will be of approximately one hour's duration, and will be illustrated by diagrams, photographs, specimens, and apparatus.

The syllabus of the lectures is announced to be as follows, and this may be taken to cover the subjects under review in a very comprehensive manner:—

> The nature, properties, testing, storage and treatment of the ingredients of Concrete—cement, sand, coarse material and water. Tools, machinery and plant.
>
> Determination of Voids and Proportioning; volume of concrete produced.
>
> Making and putting in place. Surface treatment and colouring. Forms for centering, making, erecting and removing. Machine moulding. Effect of cold and rainy weather.

Resistance to compression, tension, shear, abrasion, penetration and bearing, change of temperature, sound, electricity, permeation. Modulus of elasticity, plasticity and lateral pressure, alteration of volume in setting and hardening, friction, bond, coefficient of expansion, heat conductivity. Durability; hygiene. Applications.

There is no fee for the course; admission will be by ticket obtainable on application from the Concrete Institute.

The City and Guilds of London Institute—Department of Technology.—We have just received the programme of this Institute for the session 1912-13, containing the regulations for the classes and the examination of the candidates in technological subjects. We note with interest that under the examination syllabus for structural engineering, candidates are expected, *inter alia*, to have a general knowledge of the properties and strengths of stone, brick, cement, mortar, concrete, cast iron, wrought iron, and steel.

Further, the final examination under that subject includes a general knowledge of the principal systems of reinforced concrete construction as applied to buildings, arched bridges, tanks, piles, and conduits. It also includes a knowledge of simple calculations, the beam theory as applied to reinforced concrete beams and floors, and of materials and specifications.

Classes in Reinforced Concrete at the Northern Polytechnic Institute, Holloway.—In their session 1912-13 the Northern Polytechnic have included a series of classes for the study of concrete and reinforced concrete. The lectures at these classes will consist of the following subjects: Manufacture and use of materials; mechanics required for the subject; theory of reinforced concrete; design of floors, beams, columns, walls, etc.; R.I.B.A. Report and L.C.C. Regulations; making of beams, etc., and testing; lectures illustrated by lantern slides, steel reinforcements, and models; visits to works under construction.

The classes will be conducted by Mr. R. Graham Keevil, Civil Engineer.

An interesting lecture was held by Mr. J. S. E. de Vesian, M.Inst.C.E., in connection with opening of these classes, on "Some Recent Typical Examples of Reinforced Concrete Construction." In the course of the paper Mr. de Vesian pointed out that when Mr. Mouchel first undertook the design of Hennebique reinforced concrete, not a single example of genuine reinforced concrete construction was to be found in the United Kingdom, but since that time over 1,300 structures have been completed in Great Britain and Ireland. The total number of structures of this kind throughout the world now exceeds 24,000. The lecturer then went on to describe the theoretical and the practical sides of the subject.

The Concrete Institute's Visit to Works in Course of Construction.—By courtesy of Mr. W. Willox, M.Inst.C.E., Chief Engineer of the Metropolitan Railway, some of the members of the Concrete Institute paid a visit to the Metropolitan Railway's new offices at Baker Street, and the reinforced concrete bridge at King's Cross Station. Of the latter we gave particulars in our October issue.

Regarding the new offices at Baker Street, these are intended to supersede the offices at Westbourne Terrace.

These offices will have a front elevation of 140 ft. in length, and the height from foundations to roof will be approximately 90 ft. The back portion will have two wings, measuring respectively 111 ft. in length by 38 ft. in width and 100 ft. by 43 ft.; the latter wing is connected to the booking hall of the new station by a footbridge 82 ft. in span, 10 ft. wide, and with a height of 15 ft.

A retaining wall in reinforced concrete, and of a length of 152 ft., will run parallel to the front elevation. The total length of the retaining wall, including the returns, will be 212 ft., the height being 24 ft.

The walls of the building are to be entirely in reinforced concrete of a thickness of 6 in., and for the principal elevation the wall is to be faced with brickwork and faience.

The building will comprise a lower basement, basement, booking-hall floor, ground floor, first, second, and third floors, and a flat roof. The superloads up to the first floor included are to be 150 lb. per sq. ft., the second and third being 90 lb. per sq. ft., and the roof 40 lb. per sq. ft. The total superficial area of floors and roof in reinforced concrete will be approximately 65,000 square feet.

The building is to be carried on concrete stanchions over the new platforms, the lowest floor being level with the booking hall of the new station; store and strong rooms, and the heating chamber being at a lower level and alongside a set of rails, thus facilitating the handling of the coal for heating, and the stores.

The whole of the work is under the control of Mr. W. Willox, M.Inst.C.E., Chief Engineer of Metropolitan Railway.

The architectural work has been designed by Mr. Willox's architectural assistant, Mr. C. W. Clark, A.R.I.B.A., P.A.S.I.; and Mr. O. G. C. Drury, A.M.I.C.E., is the resident engineer. The reinforced concrete work is on the Coignet system.

By the courtesy of Messrs. Drew, Bear, Perks, some of the members of the Institute also visited that firm's steel works at Battersea last month.

Reinforced Concrete Lining to Mine Shafts.—What is stated to be an entirely new application of reinforced concrete construction has recently been carried out at the Plenmeller Collieries, Haltwhistle, Northumberland, by Mr. T. Blandford, the engineer to the colliery company, who has adopted the system for lining two new shafts, instead of the usual brick lining, segmental slabs of concrete reinforced with expanded steel being used. These slabs, which, says the *Journal of the Society of Arts,* were cast in moulds on the site, were made of a size (3 ft. 1 in. by 1 ft. 6 in.) convenient for handling, and, after being allowed to season, were lowered into the shaft and fixed in position. The lining of the shaft with such slabs was completed to a depth of 12 fathoms some time ago, and it has since been used under ordinary working conditions. It is reported as having proved economical in construction and satisfactory in use; so that further lining of a similar nature is now in progress. The slabs are 5 in. in thickness, and are reinforced near the concave face with expanded steel, which has a cross-sectional area of ·25 of a square inch per foot of width short-way of mesh. The slabs are tongued and grooved on edges to allow of their fitting and keying into each other, the joints being filled with cement grout to make the segments a monolithic cylinder when placed in position. A hole was left in each slab—about 1 in. in diameter on the concave face, and slightly larger on the convex face—to allow of their being slung and lowered into position, as well as to serve as an inlet for cement grout to be injected into the concrete filling behind them after the slabs had been fixed in position. The lining of the shafts was completed through water-bearing strata which had been previously treated by the François method of cementation, and the cavity between the reinforced concrete lining and the face of the strata, left in the process of sinking the shafts, was filled with ordinary concrete to a thickness of 9 in. behind the lining during the course of the fixing of the lining; this concrete being finally filled up solid by injection through the lining with cement grout, thus making it watertight. It is said that the new lining possesses certain advantages over the ordinary brick lining, as being more watertight, and as not requiring any outlay in upkeep once the work is completed.—*Page's Weekly.*

Waterproof Concrete.—Some tests carried out by the United States Geological Survey and the Bureau of Standards on Portland cement in order to determine its permeability to water show that the concrete is less permeable in the case of mixtures rich in cement, and that the permeability increases with age. The permeability, however, is not dependent entirely on the quantity of cement used, but is also affected by the ratio of coarse to fine aggregate employed, for concrete made with a sand containing a large percentage of fine materials was less permeable than that made with larger particles. In general the experiments have shown that Portland cement mortar and concrete may be made practically waterproof at any hydrostatic head up to 40 ft. without the use of any "integral" waterproofing materials, of which about forty different kinds, purchased in the open market, were tested. But in order to obtain such impermeable concrete considerable care should be exercised in selecting good materials as aggregates and in proportioning them in such a way as to obtain a dense mixture which shall be sufficiently wet to enable it to be puddled and to allow the particles to flow in position without being tamped. The addition of waterproof compounds does not compensate for lean mixtures, for poor materials, or for inefficient mixing, nor do they have much effect on permeability. None of the compounds reduced the absorption materially before the samples were dried by heating at 212 degrees F., but some did decrease the absorption after the samples were so dried. None seriously affected the compressive or tensile strength in the quantities used in the tests, but the addition of inert void fillers up to 20 per cent. of the volume of the cement increased the compressive strength.—*Times.*

Ammonia Tanks of Reinforced Concrete.—According to the *Tonindustrie Zeitung*, little has hitherto been published on the use of concrete tanks for the storage of ammonia. Ammonia when combined with acids attacks the concrete, as in the case of sulphate of ammonia; and many works' effluents—such as those from artificial manure works, salt works, sulphuric acid and hydrochloric acid factories, ammonia works, and galvanising works—also attack concrete. It is, however, reported from America that in the year 1909 two tanks for the gasworks at Ann Arbor, in Michigan, were constructed of reinforced concrete, and that up to the present they have behaved well and are quite sound. The tanks are superposed, the larger being 40 ft. by 10 ft., and the smaller 30 ft. by 8 ft. Both were 13 ft. deep. The available capacity of the larger one is 23,000 gallons, and of the smaller 14,000 gallons. The soil was a shifting sand, from which the water was pumped during construction. The outer layer of concrete was 2 in. thick, the ballast or filling being not larger than 2-5 in. After scraping and cleaning, the inner walls were rendered with a mixture of one part of cement to two of sand. Both the bottom and sides were reinforced crosswise with iron.—*Journal of Gas-Lighting.*

Laying Concrete During Frost.— Interesting particulars are given by the *Municipal Journal*, of New York, of a system of laying concrete in freezing weather. The account is contributed by Mr. E. H. Collins, city engineer of Liberty, Mo. He says:—"The contract called for the pavement to be finished on or before November 15th, but freezing weather set in unusually early, and on November 1st there still remained to be placed about 1,200 ft. of street, or 4,000 sq. yds. of pavement. It seemed very desirable to clean up the street, and, consequently, concrete was placed whenever opportunity offered and it was not too cold, ½ lb. of salt to the barrel of cement being placed in the wearing surface mortar, and the day's work being covered each evening with bedding straw and litter from livery barns. The sub-grade was also kept covered with the same kind of material to keep it from freezing, this material being removed immediately ahead of the concrete and the ground swept clean." The writer adds:—"In order to test the effect of freezing on this construction, some forms were made 8 in. by 12 in. by 6 in. deep, which were filled with concrete, and these blocks left on the street until it was opened for traffic; and, although the mercury went down nearly to zero during some of the nights following the placing of the concrete, these samples showed up as strong as samples taken in October, the salt and manure apparently having saved the concrete; and at this date the pavement gives no evidence of weak spots and not a crack can be found, although we have just passed through the most severe winter we have had in twenty years." The following is a description of the manner in which the pavement was constructed:—"The sub-grade was thoroughly rolled with a heavy roller to a compact surface and a 5-in. concrete base placed thereon, composed of 1 part Portland cement, 2½ parts sand, and 5 parts broken stone, or 1 part cement, 2 parts sand, and 5 parts coarse Joplin grit, 'with sufficient water to make a concrete of such consistency that when rammed the mass will not shake, but will, when struck, compact within the area of the face of the hammer without displacing the material laterally.' This concrete was rammed until free moisture appeared on the surface, and a wearing surface was immediately placed, consisting of 1 in. of mortar composed of 1 part cement and 2 parts sand. This wearing surface was steel-troweled and finished with a wood float, and before it had hardened was brushed with a stiff brush or broom where the engineer directed. The pavement was left in a finished condition every evening, even if the finishers had to work overtime."

Reinforced Concrete Wharf and Jetty at Port Talbot Docks, South Wales.—
—The Port Talbot Railway and Docks Company have recently received schemes and tenders on several of the principal systems of reinforced concrete for a wharf and jetty for Port Talbot Docks.

After careful investigation of the various competitive schemes by Mr. W. Cleaver, M.I.C.E., engineer of the company, it was decided to accept the tender presented by Messrs. Watt Bros., of Cardiff, licensees of the Coignet system.

The scheme comprises a jetty for coal belt-tipping appliance, having a frontage of 65 ft. and a width of 45 ft., the deck being supported by three rows of piles.

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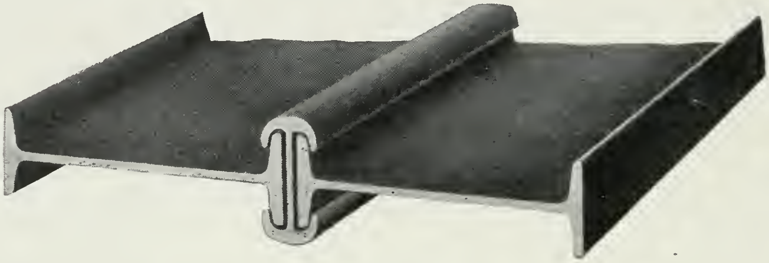

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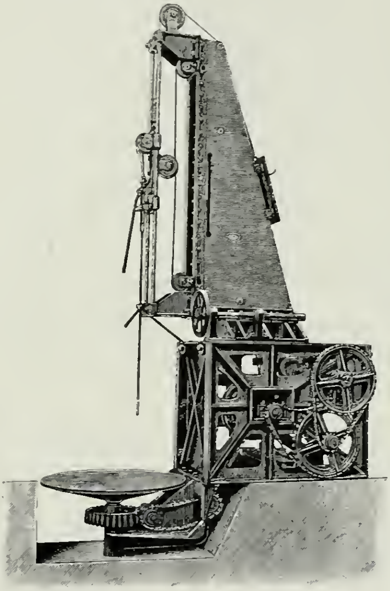
The wharf has a frontage of 600 ft. length with a width of 45 ft., and approaches 145 ft. long at each end. The deck in this case is supported by four rows of piles strongly braced together. The permanent depth of water in each case is 32 ft., with decks about 7 ft. above the permanent water level. The work has been calculated to carry three railway lines for heavy locomotives and one crane road.

The scheme for the reinforced concrete work has been prepared by Messrs. Edmond Coignet, Ltd., of Westminster.

Santos Drainage System.—The six canals which will form the new surface drainage system of Santos are being constructed entirely of reinforced concrete. The first three are completed, and the fourth is now in course of construction. The other two have not yet been commenced.

TRADE NOTICES.

Concrete Pipe Stamping Machines.—The accompanying illustration shows a concrete pipe stamping machine. The machine is simple in construction, consisting of three parts: (*a*) a revolving slab, (*b*) a metal upper frame, and (*c*) a metal underframe. The stamping of the pipes is done automatically by means of very rapid short strokes, and the only manual labour involved is the placing of the cement into the form.



The makers claim that the initial cost of installation will repay the manufacturer by the increased turnover he will obtain in using the machines, as also in the saving of materials and labour. It is stated that the pipes produced by this machine are able to resist a considerably higher pressure test owing to their homogeneity, a quality which it is not possible to obtain in hand-made pipes. The machines can be easily installed and put into action by the manufacturer. They are made in three sizes, and require 2 h.p. for working them. Full particulars and prices can be obtained from Ferd. Wienberg Engineering Works, Copenhagen, Denmark.

The Lock Woven Mesh System. A very interesting booklet has recently been published by Messrs. George F. West and Co., of Caxton House, Westminster, S.W., regarding the above method of reinforcement. The booklet is not only well illustrated, but contains much useful information. Considerable space has been devoted to questions such as materials and specifications, and the booklet also contains a number of useful tables and calculations. It can be obtained from Messrs. West at the above address.

Constable & Co., Ltd.—We would refer our readers to a highly interesting article on the "Modern Railway Terminal," which appears in *Scribner's Magazine* for October. The journal is published by Messrs. Constable and Co., Ltd., 10, Orange Street, W.C.

ENQUIRIES.

DEAR SIR,—For some time past I have been trying to find out the temperature at which Portland cement sinters, and more especially to know if it can be melted, and if so, at what temperature?

From reading the literature on the subject I have not been able to learn whether it melts or not.

If you could kindly furnish me with answers to any or all of the above questions I shall be very grateful.

Thanking you in anticipation of a reply,

I remain, yours faithfully, C. P. W.

The Editor, CONCRETE AND CONSTRUCTIONAL ENGINEERING.

Reply.—The temperature at which Portland cement of normal composition vitrifies, or sinters, ranges between 1300 and 1450° centigrade, the exact temperature being governed largely by the ratio of the acid contents present in the raw material; all the lime-silica compounds fuse above 1450° centigrade.

For further particulars on this subject reference should be made to the works of Messrs. Day and Shepherd, of the Carnegie Institute, Washington, who have done considerable work in this field of inquiry.

EDITORIAL MEMOS.

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DATES OF ISSUE.—This Journal is issued Monthly. For Advertisement Rates apply to Publisher. Matter for displayed advertisements required by 12th of month prior to subsequent issue, for Wants column by 18th.

SUBSCRIPTION RATES (Prepaid.—Post free).

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Address remittances to Concrete Publications, Ltd.

General Offices:
North British and Mercantile Building,
Waterloo Place, London, S.W.

Telegraphic Address: "Concretius London."
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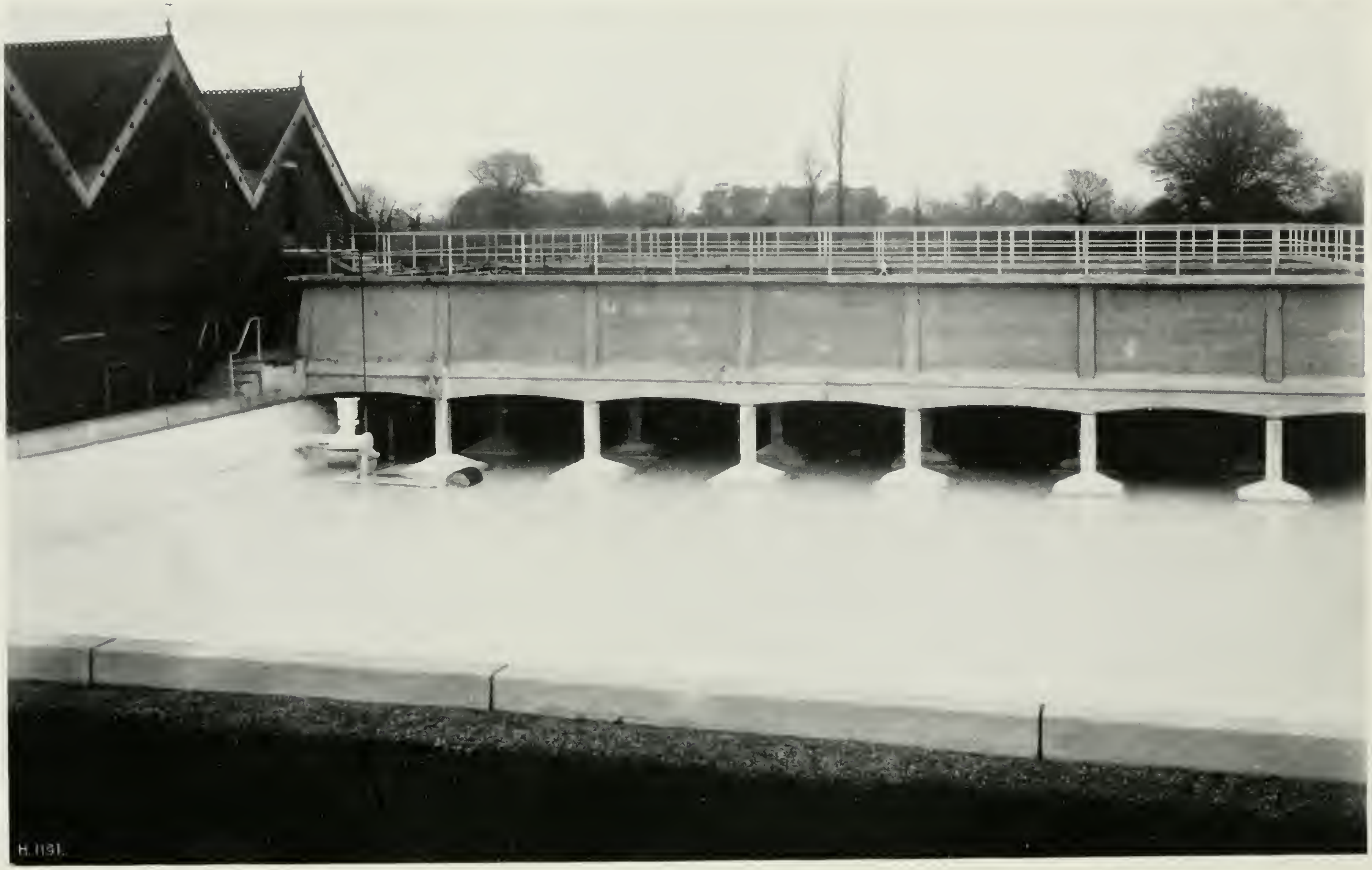


(*For description see page 895.*)

Elevated Reinforced Concrete Lime-Water Tanks for the Frimley and Farnborough District Water Co.

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

Volume VII. No. 12. LONDON, DECEMBER, 1912.

EDITORIAL NOTES.

THE RESPONSIBILITY OF THE LONDON COUNTY COUNCIL AS TO SAFETY OF LIFE FROM FIRE.

IN 1905 the London County Council obtained an Amendment Act to the London Building Act, with the view of safeguarding the public from fire, and this Amendment Act gives the Council certain useful powers as to means of escape and other matters having direct bearing on the safety of life in a large number of the older buildings in London.

The Amendment Act—which was well thought out, and thoroughly discussed in Committee in the House of Commons before it was placed on the Statute Book—is in many ways an excellent measure.

This Act, as has been indicated, has now been on the Statute Book for no less than seven years, and allowing a year for all the preliminary routine measures that had to be taken upon its adoption, there have been at least a solid six years in which active work could have been done energetically and extensively under its rights. As a matter of fact, the buildings that come under this Act have been carefully scheduled, and we speak with authority when we say that no less than *fifty thousand buildings* were, as a matter of fact, scheduled as requiring the attention of the London County Council under its new powers. But of these *fifty thousand buildings* less than fifteen hundred cases have been energetically pressed, and, if we are rightly informed, under six hundred have been remedied.

To put it quite bluntly, in the six working years that have passed over 49,000 buildings have failed to receive that attention which is required in the interests of public safety.

This, to our mind, is one of the gravest scandals in the annals of the London County Council, and we here speak of that body in its corporate capacity, and its Building Act Committee in particular.

We exempt from all possible blame the permanent officials of the Council, that is to say, their admirable Superintending Architect, Mr. Riley, and his two chief assistants concerned in Building Act matters—viz., Mr. Capon and Mr. Stransome. These gentlemen were the primary advisers in the drafting of the Building Acts Amendment Act, and it is well known that they have the safety of the public at heart. The safety of London has been increased materially under Mr. Riley's régime, and we consider that it is disgraceful that he should not have been accorded the necessary staff and the necessary facilities and active encouragement, to carry out the work that it is the duty of the Council to see duly executed, and which would enable him to put London more properly in order from the fire point of view.

The matter is of such serious gravity that a copy of this journal will be transmitted by registered post to every member of the London County Council, so that there may be no future excuse as to the matter having escaped the notice of any one member. We shall also send a copy to each magistrate, coroner, and other public officials concerned, so that the Council's *rôle* in this matter may be properly appreciated.

Perhaps this may lead to the Superintending Architect being authorised to carry out the necessary work with his wonted thoroughness and promptitude. If not, there will be no difficulty in future to allot the blame when fatalities occur in buildings that do not show efficient safeguards.

BELGIAN CEMENT.

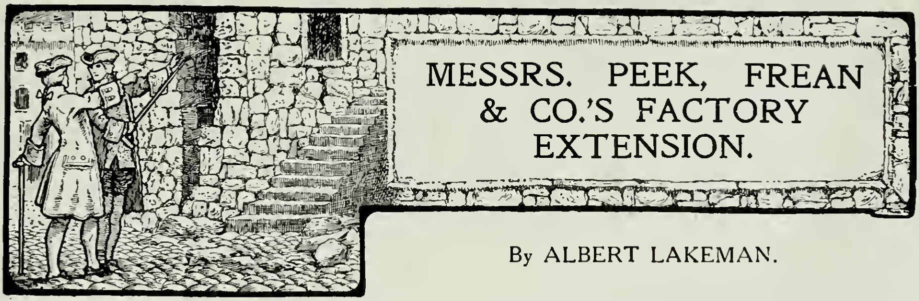
In a recent issue of a financial contemporary there appeared a paragraph headed "Belgian Cement Increasing its Hold on the World's Markets." As we are not in the secrets of its Editor's sanctum we have no knowledge of the origin of the paragraph referred to, but would hazard a guess that somebody interested in the sale of this commodity furnished the data on which this notice was founded, and it is fairly obvious that the Editor himself was quite unfamiliar with what has been going on in the cement world this year. As the figures undoubtedly reveal a considerable increase in the Belgian exports of cement during the first nine months of 1912, it is not surprising that anyone unacquainted with all the circumstances should see in them strong evidence of virility in the Belgian cement trade, but without wishing to depreciate the importance of this phase of Belgian industry, it is desirable to place it in its true perspective lest it should be thought that our English cement makers are falling behind in the race for supremacy in the world's markets.

The facts are that not only in this country, but generally throughout the world, the consumption of cement is steadily increasing. New uses are being found for cement from day to day, and the growing popularity of reinforced concrete for constructional purposes continually strengthens the demand. Although new works are gradually springing up in countries which have until recently depended entirely on Europe for their cement supplies, the European exports of cement to the British Colonies, to India, and especially to South America have been growing rapidly during the past three years. A careful comparison of the export statistics for the three great exporting countries during these three years shows that England led the way in 1910 and 1911. Her exports of cement, despite the fact that freight rates from London were often higher than from Antwerp or Hamburg, increased more than the exports from either Belgium or Germany. It was not until the English cement industry became paralysed by the great strikes of 1912 that she lost her leading place in the race. For almost three months out of the nine the great cement industry of the Thames and Medway was brought to a complete standstill. The coal strike of March led to an exhaustion of fuel supplies, and before arrears of deliveries could be brought level again the works were once more paralysed by the Transport Workers' strike, and for weeks on end not a wheel was turning in the mills. It is computed that the loss of output caused by these two strikes amounted to no less than 400,000 tons on the Thames and Medway, whence almost the whole of the British export trade in cement is carried on.

Not only were the London cement makers unable during the strike periods to supply their export customers, but they were unable even to supply the home trade. Though consumers as a whole were considerate, they were not all content to wait, nor to see important work brought to a standstill without a desperate effort to obtain cement elsewhere. In the end the German and Belgian cement makers reaped the benefit of the industrial warfare in our midst. They, and they alone, were able to supply cement to those foreign markets which had obtained their cement supplies from London, and the trade was diverted to them, not by any superiority of their products, but solely by reason of the temporary extinction of British competition. Of course their exports increased. It was not even necessary for them to bestir themselves. Like Matthew, they needed only to sit at the receipt of custom and supply cement to the hungry consumers who came thundering at their doors demanding anything and everything which bore the name of "cement," regardless of quality and almost regardless of price. Not only to foreign markets, but also to the United Kingdom itself, cement was shipped from Hamburg, Antwerp, and Rotterdam in haste and brought into use without preliminary tests of any sort. Anything was better than nothing when the choice lay between buying abroad and a complete stoppage of work, with all its attendant inconveniences and loss. The imports of cement into the United Kingdom rose from a monthly average of less than 4,000 tons during the first quarter of the year to one of about 8,000 during the second quarter, then jumped up suddenly to 24,544 tons in July and 22,707 tons in August, but fell to 12,663 tons in September and 10,194 tons in October. People no longer wanted the foreign article when the Thames and Medway works got once more into full swing, and could resume deliveries both at home and abroad.

Before leaving the subject it will be well to remind our readers of what has already been alluded to in our columns—viz., the true character of the so-called "Best Portland Cement" sent into the United Kingdom from Belgium. Good, genuine artificial Portland cement is undoubtedly produced in Belgium, but is usually no cheaper than the genuine Portland cement produced in Great Britain. The Belgian producers of the genuine article have been fully occupied throughout the year with the supply of their normal export market and of the many important undertakings now going on in their own land. The great alterations in the dock and wharf accommodation of Antwerp are consuming large quantities of genuine cement, and so also are the new fortifications which will replace those destroyed or rendered obsolete by the port works in question. The increased Belgian exports to Great Britain have not consisted of genuine Portland cement, except to an infinitesimal degree. The increase is in the import of that material which is known in the trade as "Natural Cement," is notoriously inferior, utterly unreliable, and often even dangerous. The prudent professional man will never allow its use on works under his control. Natural cement is nothing more than hydraulic lime, and goes through nothing more complicated in the course of its manufacture. There is no careful mixing of two separate raw materials, with all the refinements of scientific control at every stage of the process and at every hour of the day in order to remedy the ever-present variations in the chemical composition of the limestones, shales, clays, marls, etc., employed, and secure a uniform chemical composition of the resulting product, by which alone a uniform quality can be obtained. The rock, which varies greatly in its chemical composition and is usually deficient in lime, is taken just as it comes from the quarry, burned and ground exactly as if it were to be sold as "Hydraulic Lime." Indeed, much of it is sold under that name in its own country. But the wily vendor is keenly alive to the value of a name, and knows by experience that many persons who would not touch it under that description will buy readily enough if it be sold as "Portland Cement," or "Best Portland Cement," and to give an extra touch of realism, he packs it in a cask exactly like that used for genuine Portland cement, is ever ready to attach any label preferred by the purchaser, and more often than not he prints his label in the English language, calls his firm by an English name, and does everything he can to make the guileless consumer imagine he is buying genuine artificial Portland cement produced by British makers, whose reputation throughout the world still stands as the surest guarantee of high and reliable quality.

Stimulated by the cement famine created in the United Kingdom by the great strikes of the present year and the temporary paralysis of the great centre of the English cement industry, the agents of the Belgian producers of "Natural Cement" have undoubtedly been able to place more of this inferior material on our home market, and the possibility of making a higher profit on its re-sale has undoubtedly tempted certain builders' merchants to push it more actively than during the last few years, but these facts make it necessary for engineers and architects to exercise special vigilance to prevent its substitution for the genuine article.



MESSRS. PEEK, FREAN & CO.'S FACTORY EXTENSION.

By ALBERT LAKEMAN.

The extension to Messrs. Peek, Frean's factory contains many interesting features, and we give some particulars of same in the following article.—ED.

The extension of Messrs. Peek, Frean & Co.'s factory consists of a new building in Keetons Road, which occupies a site measuring about 128 ft. by 50 ft., and it has six floors in all, including a basement. Although the external walls are of brickwork, the main constructional features are of reinforced concrete; in fact, it may be said that the latter material has been used as far as the present Building Acts will allow of its employment in such structures. The adoption of reinforced concrete enabled a considerable saving to be made in the cost of the building, and it was chiefly on this account that it was employed apart from its eminent suitability for a structure of this nature. The general disposition of the building will be seen in *Fig.* 1, which is the plan at the first-floor level. In addition to the accommodation shown on this plan extensive storage and packing rooms are provided, also showrooms, drawing office, chief engineer's room, and other offices, while the top floor is utilised as an experimental and chemical laboratory. The new building is connected to one of the existing blocks by means of a bridge, which is also in reinforced concrete, as subsequently described. The total height from the basement floor level to the roof is about 70 ft. for the main portion of the building, while a yard is provided under the lower portion of the building, as indicated in *Fig.* 2, which shows the east elevation of the new portion.

The type of foundation employed is illustrated in *Fig.* 3, the area covered in this particular example being 19 ft. by 11 ft. 6 in., and the method being practically that of a double cantilever, with a projection below the slab. The thickness of the slab is 2 ft. 4 in., and this is reinforced with Moss bars and stirrups, the latter being spaced at distances varying from 6 in. to 9¼ in. centres in the lower surface. The main cantilever or rib projects 2 ft. 2 in. below the slab and has a maximum width of 3 ft. 6 in. and a minimum of 2 ft. 6 in., with reinforcement consisting of Moss bars and stirrups. This foundation carries the load from two columns and is a strong and economical form to employ.

The columns are square in section, and some typical details are illustrated in *Fig.* 4. It is interesting to note that although the floors were calculated to sustain an external load of 2½ cwt. per ft. super., the columns are designed to carry double the normal load on all floors without any reduction being made for the supports on the lower floors, as is usually the case. The reason for this is, that it is necessary at times to store a great quantity of goods on each floor

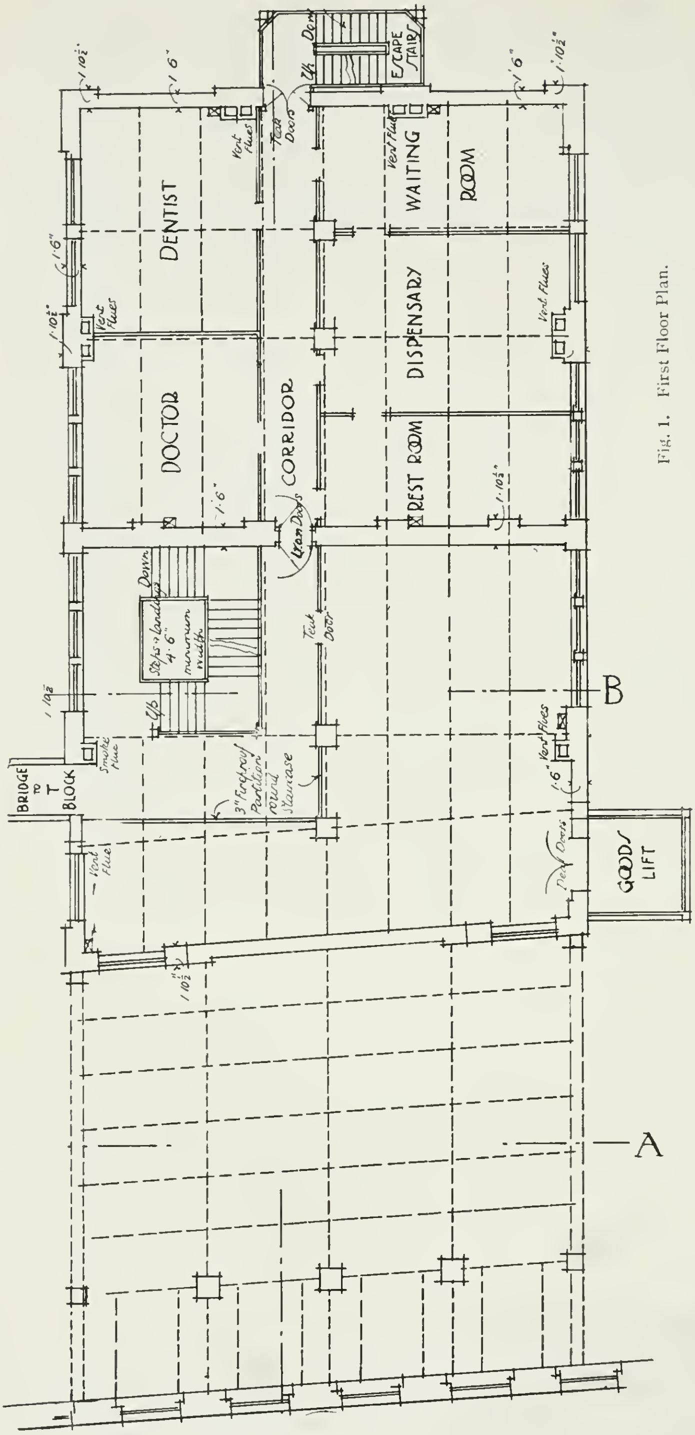


Fig. 1. First Floor Plan.

Messrs. Peek, Frean & Co.'s Factory Extension.

when preparing for shipment, and owing to the enormous output of the firm, every available foot of space has to be loaded to its fullest capacity. The details show the largest columns employed, one of these being 32 in. square in the basement and reinforced with twelve lines of vertical reinforcement, consisting of four $1\frac{1}{2}$-in. diameter bars at the corners and eight other bars each $1\frac{1}{4}$-in. diameter. Inner and outer ties were provided to link the vertical lines together, and these were spaced at distances varying from 4 in. to 9 in. These large columns are reduced in size up to the top floor, where they are 10 in. square and reinforced with four vertical rods having a diameter of $\frac{3}{4}$ in.

The floors were constructed with main beams, secondary beams, and slabs, the first-named having a span of about 24 ft. and being about 32 in. deep and 13 in. wide. The reinforcement consisted of

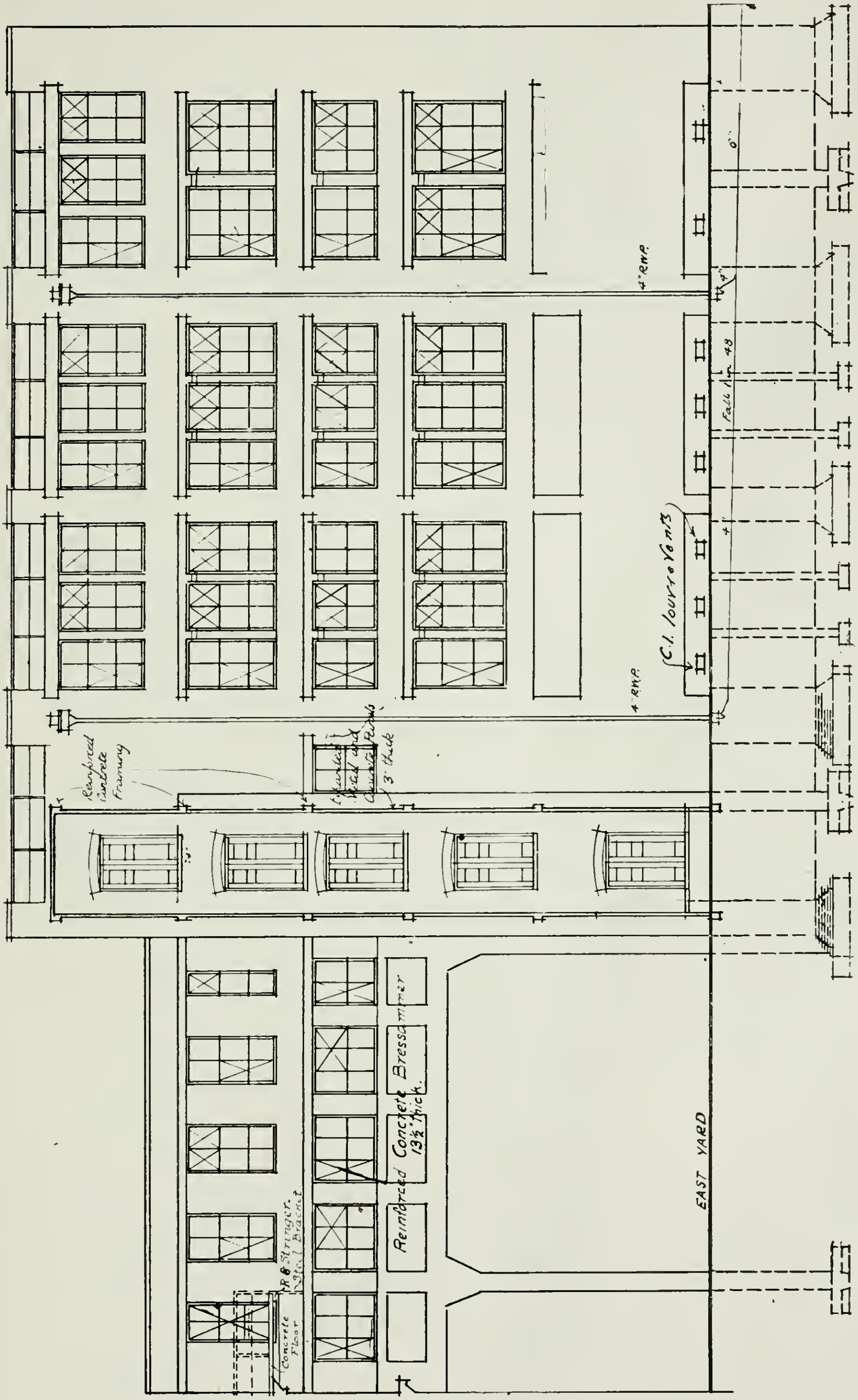


Fig. 2. East Elevation of Extension.

Messrs. Peek, Frean & Co.'s Factory Extension

Moss beams and stirrups, while the secondary beams are generally of two sizes—viz., 16½ in. by 10 in. and 13 in. by 10 in., the former spanning a dis-

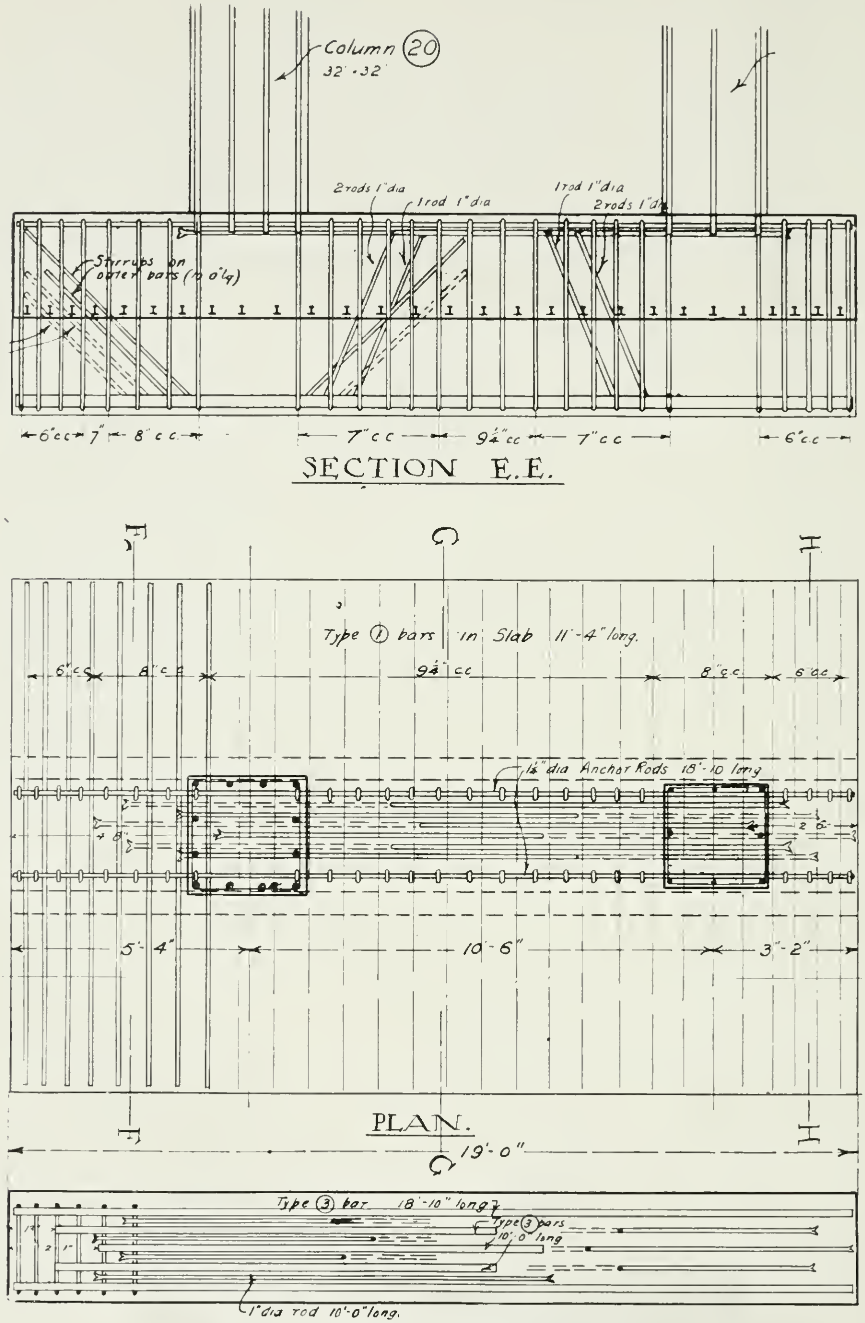


Fig. 3. Plan and Section of Foundation to Column.
Messrs. Peek, Frean & Co.'s Factory Extension.

tance of 18 ft. and the latter a distance of 8 to 12 ft., according to the position. The floor slabs are 4 in. thick, reinforced with ½-in. rods and made continuous over several spans. In the work to the roof over the annexe the main beams are 24 in. by 10 in. reinforced with two Moss bars and three ⅞-in. diameter round rods and many stirrups. The secondary beams are 13 in. by 8 in., and the slabs are 3½ in. thick, laid to falls, and reinforced with ½-in. rods at 8½-in. centres. All the lintels over the windows were constructed as reinforced beams, and the mullions were designed as short columns in order to relieve the walls as much as possible.

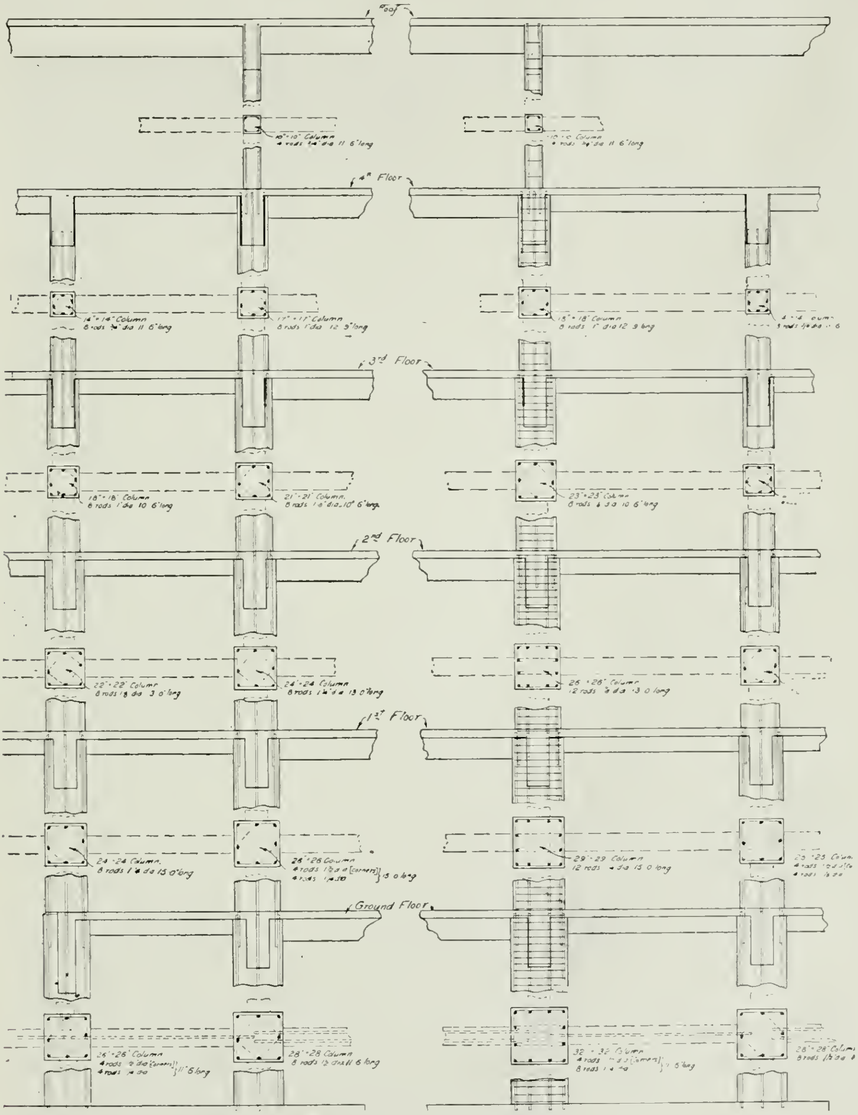


Fig. 4. Typical Column Details.
Messrs. Peek, Frean & Co.'s Factory Extension.

The bridge which connects the extension to the existing buildings is shown in the photographic views and also in *Fig.* 6, which is a drawing of the elevation. It is so constructed that a passage-way, 4 ft. 6 in. wide, is formed on the first and second floors, and it has a total height from the ground of about 37 ft.

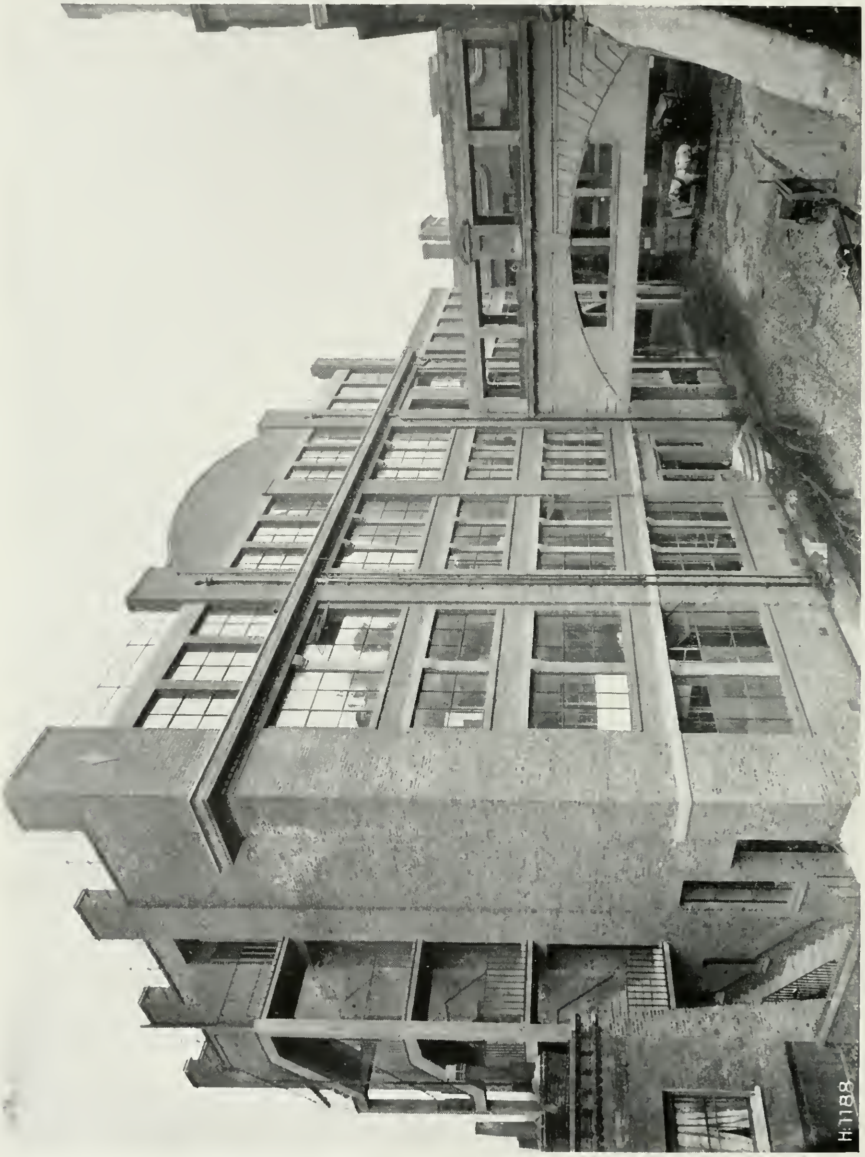


Fig 5. View Showing Bridge and Escape Stairs.
Messrs. Peek, Frean & Co.'s Factory Extension.

The walls and the arch itself are 9 in. thick and well reinforced, and even the cornice, which has a projection of 24 in., is constructed in reinforced concrete. Apart from the construction it is interesting as an example of the architectural treatment of reinforced concrete, and must be considered very effective from this point of view. The clear span of the arch is about 41 ft. and the rise is 7 ft. 10 in., the lower passage-way being on the same level as the springing.

The escape staircase is interesting, as it is rather exceptional to find reinforced concrete used for this purpose. The use of iron alone is almost universal, although it does not possess the same advantages as concrete, which is eminently fire-resisting while requiring no maintenance, and it would obviously not get so hot as metal in the event of a fierce fire. This staircase is constructed with reinforced stringers on the outside face, 16 in. by 8 in., with three $\frac{3}{4}$-in. diameter rods carried by columns 10 in. square for the upper portion and 12 in. square for the lower part, reinforced with four $\frac{3}{4}$-in. diameter rods and $\frac{3}{16}$-in. diameter links at 9-in. pitch. These columns are carried by a foundation slab 3 ft. square, reinforced with five $\frac{3}{4}$-in. diameter rods in either direction, placed in the lower surface. The flights are formed with a minimum thickness of $3\frac{1}{2}$ in. at the intersection of the treads and risers, and are reinforced in the soffits with $\frac{1}{2}$-in. diameter rods at 8-in. centres. The staircase rises to a height of 48 ft. and the flights are 3 ft. 6 in. wide in the clear. The landing slabs are

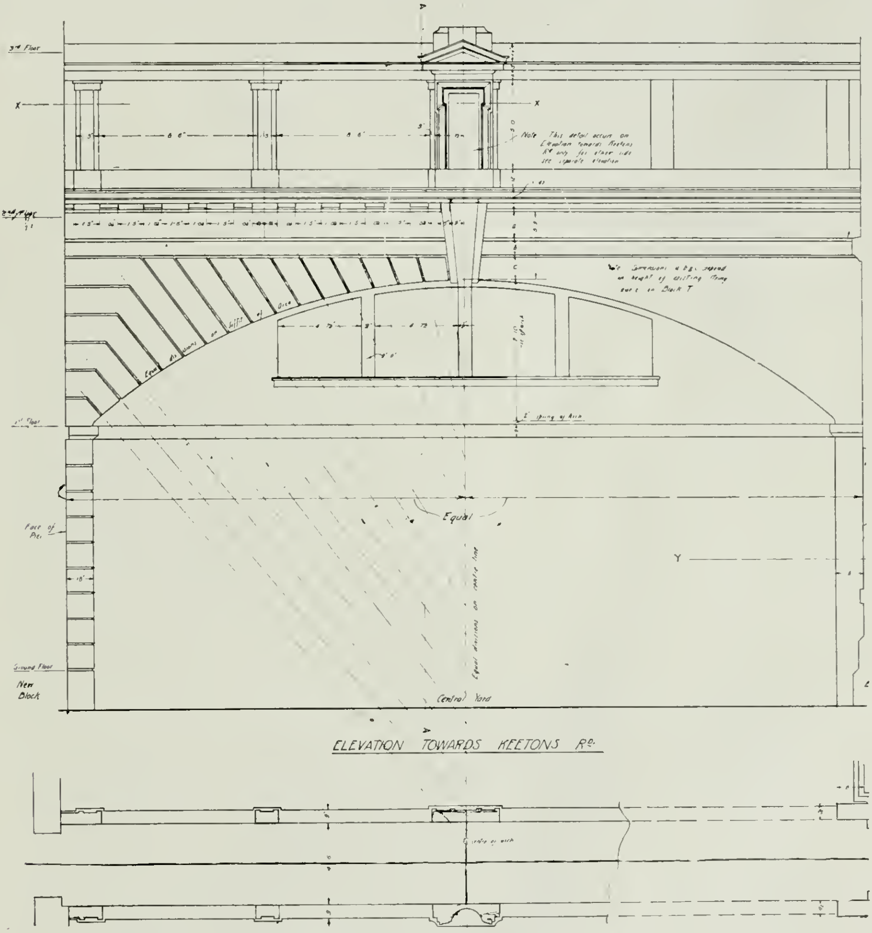

3rd Floor
X
X
8' 6"
8' 6"
Note This detail occurs on Elevation towards Keetons Rd only for other side see separate elevation
2nd Floor
Note Dimensions a b & c depend on height of existing floors over in Block T
Equal divisions on soffit of arch
1st Floor
Spring of Arch
Equal
Face of Pier
18"
Y
Equal divisions on centre line
Ground Floor
New Block
Central Yard
ELEVATION TOWARDS KEETONS Rd
Centre of arch

Fig. 6. Elevation of Reinforced Concrete Bridge.
Messrs. Peek, Frean & Co.'s Factory Extension.

3½ in. thick, and one of these, situated between the ground and first floors, is

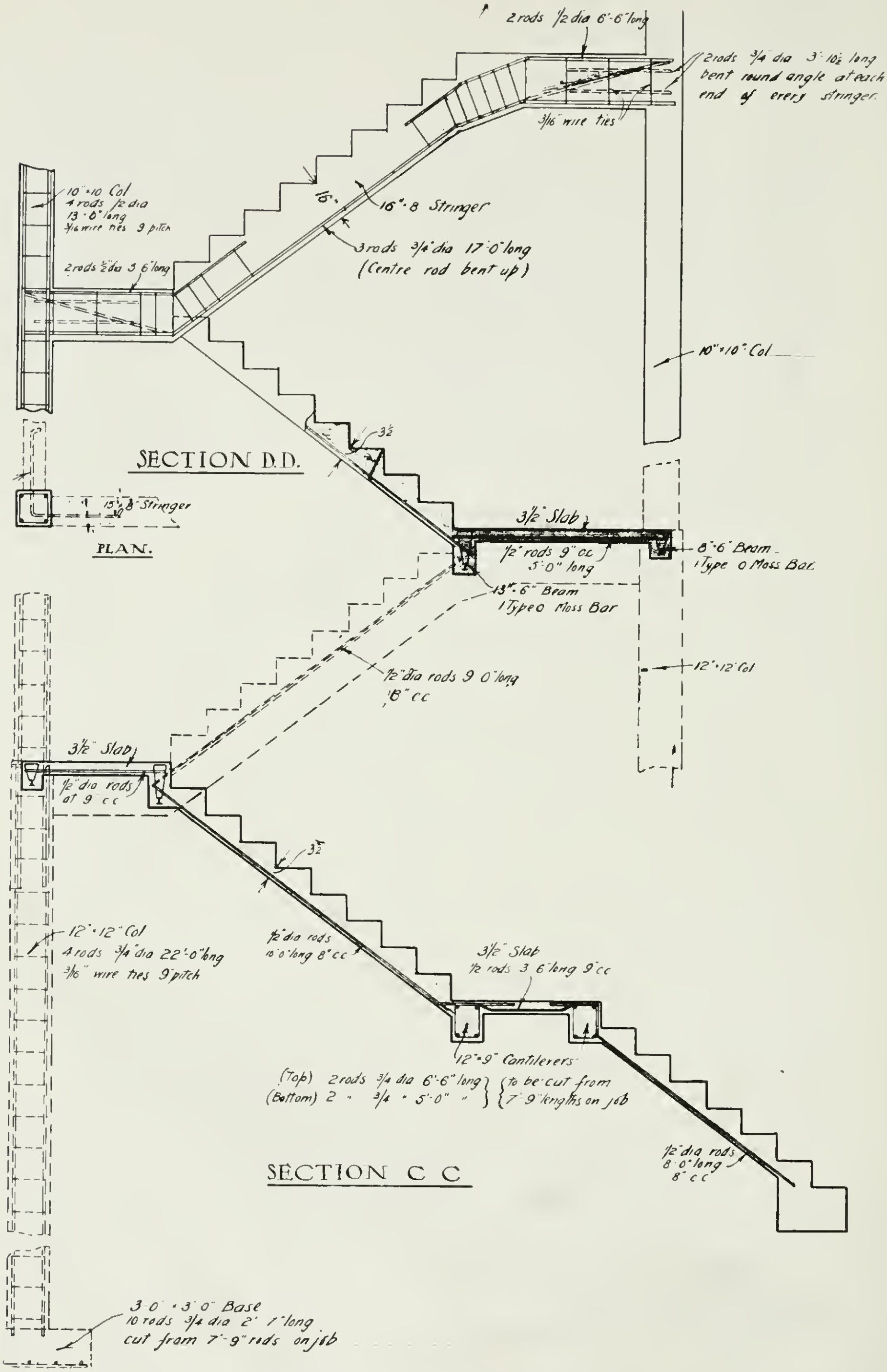

2 rods ½ dia 6'-6" long
2 rods ¾ dia 3'-10½ long bent round angle at each end of every stringer.
3/16" wire ties
10"×10" Col 4 rods ½ dia 13'-0" long 3/16 wire ties 9 pitch
16"
16"×8 Stringer
3 rods ¾ dia 17'-0" long (Centre rod bent up)
2 rods ½ dia 5'6" long
10"×10" Col
3½
SECTION D.D.
15"×8" Stringer
PLAN.
3½" Slab
½" rods 9" cc 5'0" long
8"×6" Beam 1 Type 0 Moss Bar.
13"×6" Beam 1 Type 0 Moss Bar
½" dia rods 9'0" long 8" cc
12"×12" Col
3½" Slab
½" dia rods at 9" cc
3½
12"×12" Col 4 rods ¾ dia 22'-0" long 3/16" wire ties 9" pitch
½" dia rods 10'0" long 8" cc
3½" Slab ½ rods 3'6" long 9" cc
12"×9" Cantilevers
(Top) 2 rods ¾ dia 6'-6" long
(Bottom) 2 " ¾ " 5'-0" "
to be cut from 7'9" lengths on job
SECTION C C
½" dia rods 8'0" long 8" cc
3'0"×3'0" Base 10 rods ¾ dia 2' 7" long cut from 7'-9" rods on job

Fig. 7. Details of Escape Staircase.
MESSRS. PEEK, FREAN & CO.'S FACTORY EXTENSION.

carried by two 12-in. by 9-in. cantilevers built out from the wall. Beams are provided to carry the other landings as indicated, these being generally reinforced with one Moss bar and four stirrups as a provision against shear. In the view of the finished building in *Fig*. 8 the lift enclosure can be seen, and it will be noticed that this is also formed in reinforced concrete. The use of the material for this position enabled a big saving of space in addition to being more economical in cost. It is an interesting building from a constructional point of view, and the planning and details of the various parts have been well considered. The whole of the work was designed by the engineering staff of Messrs. W. Moss & Sons, under the supervision of their Chief Engineer, Mr. de Colleville, A.R.I.B.A., and carried out by the firm entirely.

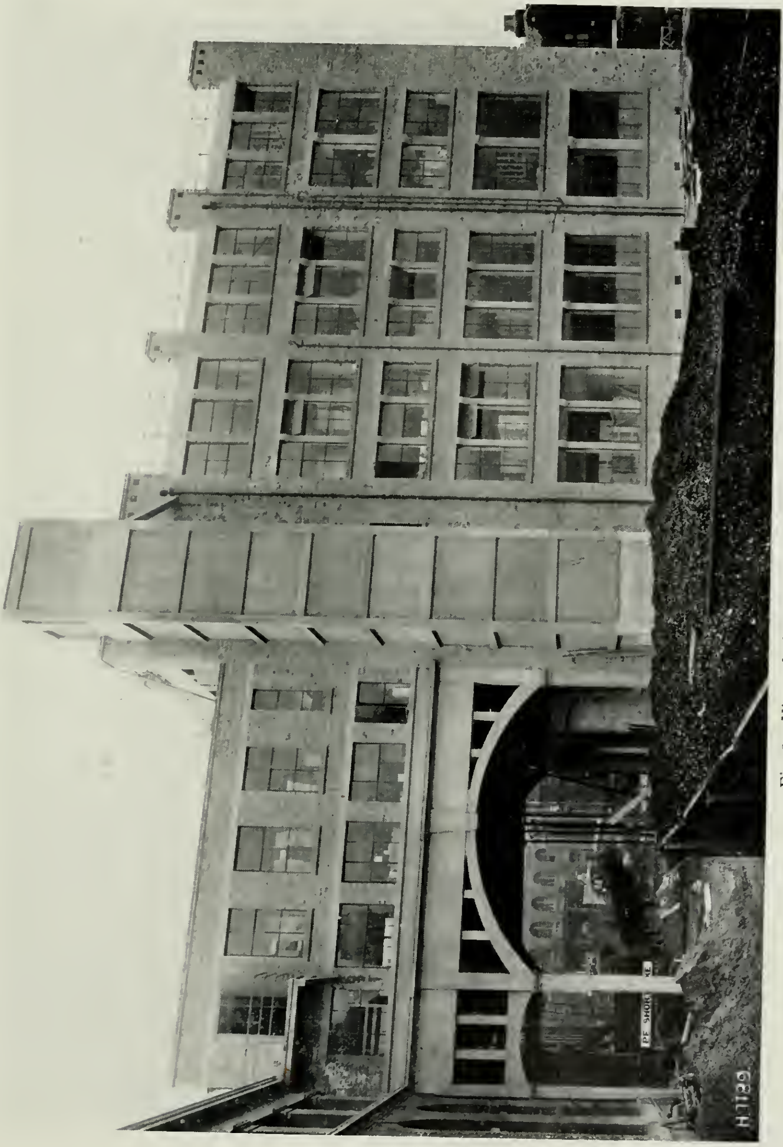


Fig 8. View of Elevation Showing Lift Enclosure.
Messrs. Peek, Frean & Co.'s Factory Extension.

THE CONCRETE INSTITUTE.

PRESIDENTIAL ADDRESS

By E. P. WELLS, J.P.

The Presidential Address with which the Winter Session of the Concrete Institute was opened was quite excellent as a practical exposition of many matters relating to concrete and reinforced concrete, and coming from one who is an authority of exceptional experience it should be carefully studied by all concerned.

INTRODUCTION.

I PROPOSE in this, my first Presidential address, to deal almost wholly with the practical side of reinforced concrete construction, as I consider that the more often attention is called to this most important part of the work the better it is for all concerned, both for the engineer, the architect, the proprietor, and also the contractor.

PORTLAND CEMENT.

With regard to the first and the principal constituent in concrete—namely, Portland cement—it is strange, at this present day, how many engineers and architects still adhere to the old specification of coarse grinding, consequently requiring aeration of the cement. It has been my lot only this week to come across a specification where it was stated the cement had to be spread on a floor for twenty-one days before being used. Those of us who are well acquainted with the present-day cement, and also its fine grinding, know how deleterious this is when great crushing strength is required. I think I have said, on more than one occasion at this Institute, that if cement is to be kept up to its full strengh it is absolutely necessary that when it is received on the site of the works it should be stored in air-tight wooden bins: and if this is done cement may be kept for many years and be just as good after the lapse of time as it was when freshly made, whereas if the cement be stored in sacks, and even a very small amount of moist air plays upon them, then the cement is rapidly hydrated and cakes in the sack. If, as is often the case, this cement be rubbed through a sieve, it becomes almost absolutely useless for purposes of concrete-making—by that I mean good concrete.

I have in my mind's eye a case of a contractor who bought cement in the month of October. The whole winter was a bad one, and he had very little opportunity of using the cement. It was stored in a shed through which the wind could blow freely, and he was rather astonished when the spring came and the cement was used that it would not set. He then wrote to the manufacturers, and when the matter was investigated the true cause of the mischief was found out.

I have myself on many occasions experimented with cement that has been so hydrated, and it is astonishing the enormous reduction in the strength; so much so that it becomes almost useless for making concrete.

Any of you who chooses to make the experiment can do so, and you will find that with the concrete made from an over-hydrated cement the amount of strength obtained will be very little indeed, and that its setting action will be so slow that it will practically take days before it shows much sign of hardening. The only use of such a cement is to mix with an over-clayed cement that is too quick setting; by this means the setting action can be retarded.

AGGREGATES.

Passing from cement, one is led on to the careful choice in all aggregates, the proper grading of the same, and also, what is of more importance, the seeing that everything is absolutely clean and the water pure.

I know it has been said that dirt is of some good in increasing the strength of cement concrete, but the only case that I can find where, over any period, a dirty aggregate increased the strength of the concrete was due to the fact that it had been mixed with a very over-clayed cement. It slowed down its setting action, and by that means did good; but if the test had been carried over a lengthy period, it would have been proved how fallacious is the advantage to concrete of dirt in any form.

With regard to aggregates consisting of gravels that are dredged either from the river or from the bed of the ocean, it is very seldom that the proper proportion of sand to coarse material is obtained. There is only one gravel that I know where one can see that the proportion is about correct, and that is obtained from the Spurn. In the ballast obtained from the Thames and along the East Coast there is at the present time a great excess of sand, and under any circumstances I would not in any way recommend that this aggregate should be used without separation of the larger particles from the finer and subsequent crushing of the coarse material.

The difference in the strength of concrete made with an excess of sand is very marked where the proportion of cement is not great; that is to say, 6 to 1 and 7 to 1 concrete with an excess of sand show very great falling off in strength, whereas with richer mixtures and an excess of sand, though there is a falling off in the early stages of hardening, after a few years the concrete will almost come up in strength to a concrete made in the correct proportions. Of course, it is well known that for all waterwork a large proportion of sand is required so as to get a perfectly dense mixture, but in no case must this be overdone.

In concrete for waterwork it is not necessary to add anything to make it watertight. If concrete be properly made it will be absolutely impervious to moisture, and if it be found necessary to reduce the labour some of these compounds of hydrated lime are, no doubt, of utility in decreasing porosity, but, at the same time, concrete so made is not improved in strength.

Before leaving the subject of aggregates I wish to call particular attention, as I have done here on more than one occasion, to the use of Fletton bricks for concrete. I have found lately that wherever Fletton bricks have been used disruption has taken place in the concrete, and I think this is due to the presence of lime in the bricks. Fletton bricks that are dangerous to use are those with purple markings, and great care should be exercised by all those who propose to use brick aggregate for concrete work to eliminate absolutely Fletton bricks in any form whatever. In this I am borne out by several who have been unfortunate enough to use the bricks, the result being that the concrete has failed.

I was called in only a few months ago to a special floor that had been put down for testing bowls, when it was found that about a month after the floor had been laid it was becoming uneven in places. On examination it was found that in every case where the floor had risen Fletton bricks in the aggregate were the cause of the mischief.

CONCRETE MIXING.

It is always advisable in the making of concrete that it should be got into the work as quickly as possible, especially in the summer: with low temperatures it is not of such great importance, as the cold slows the setting action and also its hardening. In frosty weather the drier the concrete and the more ramming, so long as there is no chance of displacing any of the reinforcing, the better for the concrete, as it will set very much more rapidly than if there is an excess of moisture.

With regard to reinforced work it is far better that the concrete should have a slight excess of moisture than a deficiency. Certain experiences, that I have had lately, have shown that where steel has been put into concrete that was too dry, the air and moisture had got through the porous concrete to the steel and caused rapid corrosion.

In one case the bottom layer of concrete had been put in and allowed to set, the steel was placed on the dry concrete, and other dry concrete was placed on top. When this work was broken up it was found that the rods in the concrete were quite loose, could be turned round by hand, and the adhesion between the two layers of concrete was anything but good. Now, had the concrete been made wet, there would have been no separation between the steel and the concrete, it would not have been possible to twist the steel rods round in the concrete, neither would moisture have passed through the concrete to the steel and caused oxydisation after the alkaline salts had disappeared by age owing to the presence of air and moisture.

Excess of moisture, as we all know, decreases the crushing strength of the concrete; but at the same time in nearly all structures there is generally such an excess of concrete in the compression member, especially where T-headed beams are used, that it is better to have the steel perfectly protected with a slightly weaker concrete than to have imperfect protection with a stronger concrete, because if imperfectly protected, corrosion will be almost bound to take place with consequent disruption of the concrete.

Excessive ramming in concrete is not at all necessary where it is made wet. It is far better to employ very light ramming and see that fine particles of the concrete are brought to the surface of the shuttering boards by means of steel slices or trowels. If this be done there will be very little danger of air finding its way through into the steel reinforcing, but a coarse aggregate first put in with an insufficiency of sand is almost always certain to have some porous parts through which the air and sulphur, especially in a London atmosphere, can attack the steel.

It may be well here to call attention to what I consider to be a most important point in reinforced concrete construction—namely, that as the concrete is being put into the work test-cubes should be made both for ascertaining the strength of the concrete *in situ* and exposed to the ordinary atmospheric conditions, and also to ascertain in the laboratory the strength of a series of cubes made at the same time and kept under laboratory conditions. One set of cubes should be kept on the works exposed to the varying atmospheric conditions, and the other at the laboratory at the normal temperature, say, of 60° F.

Some years ago I made an experiment to see whether cold had any effect on concrete, and, if so, to what extent. I found that in the month of December, when the cubes were taken from about 60° F. and placed on a roof where the temperature fell to below freezing-point, a most alarming decrease took place in the crushing resistance of the concrete, and this remained so until such time as the weather became warmer, when the crushing resistance went up and was practically the same as shown by cubes made to the same gauging but kept under normal conditions.

This clearly shows that if a building is constructed in cold weather the crushing resistance of the concrete cubes kept on the works will be low, but if it be found that the other set of test-cubes, kept under laboratory conditions, shows a rapid increase in the strength, then it is only fair to assume that the concrete that has been exposed to the cold air will, with favourable conditions, increase in strength and attain the same strength as the laboratory experiments.

The reason I am calling particular attention to this point is, that if a building is constructed in the winter and the test loads are applied up to 50 per cent. in excess while the weather is cold, there may possibly be an excessive amount of deflection, owing to the fact that the strength of the concrete in compression is low, whereas if the experiment were held over until the warmer weather, when the compressive strength of the concrete would be largely increased, then the deflection would be practically nothing—of course, assuming that the work in the first instance had been properly designed.

TESTING.

If the work is properly designed, and if the supervision has been strict and the work has been carried out according to the drawings, it is fair entirely to dispense with the testing of the structure so long as the experimental cubes show that the concrete has attained the strength required by the engineer or architect.

It is rather foolish to test a structure when one knows by experience that the crushing resistance, owing to the cold, is low. It would be far better to wait until the weather had become warmer and the concrete had parted with a lot of its moisture, when it would have become much harder. On the contrary, if it is found that the laboratory tests are low—say, for the sake of example, one month after being made—then I should strongly recommend testing of the structure to ascertain whether there were anything seriously at fault. Having those conditions of a low laboratory test and a much lower *in situ* test, yet the structure not exhibiting any serious deflection, then the work may be passed; but still, it is advisable in all cases to watch the crushing experiments, as upon these depends almost entirely the strength of the structure. A good crushing resistance means that the concrete is strong in tension, strong in adhesion, and strong in shear, and such being so, there is very little danger of the structure failing even if there is a deficiency of steel.

The testing of floors and other works is, as a rule, carried out on much too small a scale. Testing should be spread over an area that will at least take in always two sets of secondary beams as well as two spans of main beams completely. By this method of testing, the beams always have their full load and the adjoining beams become unloaded, which gives the most severe form of test, unless absolute continuous construction be carried out.

My experience has taught me that where concrete is good—that is to say, mixtures of 5 to 1 and richer—with the ordinary normal loading for which the structure was designed, unless most delicate instruments be used there will be no deflection recorded, whereas when weak concretes are used—namely, 6 to 1 and under—then the deflection at times becomes very great.

In all cases in making deflection tests, they should be made not only at the centre of the span but also at the walls, as very often it is found that a large amount of the so-called deflection is due entirely to the squeezing of the brickwork, owing to the fact that the reaction has not been spread over a sufficient area of brickwork. In no case ought the load on the brickwork to exceed eight tons per square foot; by this means a good distribution is obtained and the brick walls are strengthened thereby.

SHUTTERING AND STRUTTING.

In a great many works that I have seen where failures have taken place, the failures have been very largely attributable to shuttering and strutting. I have in mind cases of beams deflecting two or three inches after the concrete has been filled into the mould, this being due entirely to the strutting sinking into the soft ground underneath. In several cases this has caused what appeared like shear cracks at each abutment, though the work afterwards stood the test load satisfactorily. Still, such cracks made the beam look unsightly, and there could not have been the same adhesion between the concrete and steel, owing to this settlement, as if the boxes had remained perfectly true and level during the whole time the concrete was being put in and until the same were struck.

I know of one case of a failure taking place entirely owing to the strutting sinking into the ground, the sinking taking place over more than a week. The concrete, therefore, never had the least chance of getting a fair adhesion to the steel, but was constantly being drawn away from it by this settlement of the strutting, so much so that when the strutting was removed the whole structure came down with a run.

It is always advisable, if there is any doubt at all about the ground, to have the latter tested beforehand, and it is far better to increase the sole-plates to double the size required, so as to prevent any possible chance of settlement, than to have an insufficient area with consequent settlement and trouble taking place.

Not only does the settlement cause a reverse camber of the beam, but it also gives a fall in the floors the wrong way, and it needs extra expense all round to make good such defective work.

All these are points that are easily obviated by the simple means of closely watching the work as it proceeds, and not allowing any concrete to be put in until the engineer or his representative is satisfied that the strutting is of such a nature that settlement is not likely to take place.

Strutting should always remain up as long as possible after the concrete has been placed in position; in fact, if it were not for the exigencies of trade and also the rapidity with which building works of the present day have to be erected, I should personally like to see all strutting remain up for very much longer periods than is generally allowed, because the harder concrete can get before removal and any weight can get thrown upon it the better it is for the structure as a whole. Unfortunately, in the present day the question of expense has to be very largely considered, and to allow strutting to remain up as long as one would like it would increase in a great many cases the cost of the work, and I am afraid it would become almost prohibitive. Of course, in cold weather strutting must be left up for periods 50 per cent. longer than is allowed in the summer-time, and a great many of the failures in this country, and more especially in America, are mainly traceable to the removal of the shuttering and strutting at too early a period in the life of the concrete. The concrete at the time of striking was weak, but had an extra fortnight or more been allowed for the hardening, then the works that have failed would no doubt have stood up.

WATER.

Before finishing with concrete-making I would like to refer to water to be used therein. In this country, as a rule, water is almost invariably obtained from what is called a domestic source, namely, water supplied by large public companies who are extremely careful in what is sold to the public. Such being so, it is very rarely in England that one has to use water that does not come from the public supply; but there are cases where it is advisable carefully to examine the same before it is used. There are some places where the water is highly charged with gypsum compounds, and such being the case, it behoves one to see that there is no likelihood of failure taking place owing to an excess of this compound.

In the South of France, in Algeria, and in a great many districts waters are highly charged with gypsum, and so bad is this in places that concrete is absolutely dissolved and disrupted in less than two or three years. Whenever there is any doubt as to the quality of the water to be employed an analysis should be made beforehand, and so prevent what might lead to a disaster.

POSITION OF REINFORCING MATERIAL.

One of the great defects with regard to reinforced concrete is constantly raised by those who have not had much to do with the subject and often by those who are largely connected with it, namely, the difficulty of getting steelwork placed in the position designed by the engineer. It is a difficult matter always to get the work placed in the designed position owing to the carelessness of the British workman, whose idea seems to be to get the material into position as quickly as possible, no matter whether it be right or wrong, and in a great many cases if he has an opportunity of leaving the steel out he will do so. The only way to get over this difficulty is not to allow any concrete to be filled or poured in until the steelwork had been passed by the engineer or his representative. If this be done there will be very little danger to be apprehended, because if the steel is in the correct position, if the concrete is proved to be as good as borne out by the *in situ* tests and by the laboratory tests, and if the strutting has not given way, then one may fairly assume that the work has been well carried out and will sustain all the loads for which it is designed, and that there will be no deflection, or else of such a slight nature that it is not worth troubling about. If, however, the steel be badly placed, the concrete be poor, and the strutting has failed, then there is no knowing what is going to be the result.

In a great many years of practical experience I have come to this conclusion, that even if there is a large deficiency of steel in the structure both in tension and in compression, and the concrete is of an excellent quality, there is hardly any chance of failure taking place, but if the steel is up to and even in excess of the requirements asked for, and by any chance the concrete be poor, then if an excessive load be placed upon the structure there is nothing to prevent its failing. As I have stated before, a rich concrete is strong in crushing, tension, adhesion, and shear, whereas with poor concrete exactly the reverse is the case.

A series of experiments that I carried out some years ago with some beams gave for thirty-three days' test a factor of safety of $5\frac{1}{4}$ to 1; in three years the factor was over 9 to 1. The whole of this was due entirely to a very good concrete which was absolutely homogeneous throughout, and when the beams broke they failed only at the centre, the only place where cracks developed. There was no sign of shear, and the diameters of the rods where the failure took place were, to all intents and purposes, the same diameters as when they were put in. These experiments, I think, simply show an enormous increase in the lever-arm, due to the rich concrete; in fact, in the case I now mention the lever-arm was practically the total depth from the axis of the tension members to the outside of the compression member of the beam; but even this will not account wholly for the enormous load that the beams carried, as the tests produced extraordinary stresses both in the steel and also in the concrete itself.

SKILLED LABOUR.

A great deal has been done by the London County Council School of Building in educating clerks of the works and others as to the method in which the practical part of reinforced concrete construction should be carried out. They ought to go a step lower and take in hand labourers who are connected with the carrying out of the work, because until such time as they themselves understand, to even a very slight extent, what is required of them, so long will they be careless in doing the work they have to do.

The foremen employed must be instructed to keep a sharp look-out on all their men, and not to allow laxity in any shape or form. If the men know that the foreman is up to all their so-called tricks of the trade, then they will take good care that the work is carried out in an efficient manner; but if by any chance the foreman is careless, his workmen will be the same; and it will, I think, very often be found that where anything serious takes place it is not only the men but the foremen and even the clerk of the works on the job who are answerable for this state of affairs.

Any foreman who persistently makes mistakes in the carrying out of the work, after the same has been pointed out to him, should be dismissed and not allowed to undertake any work of the kind in future. It is only by making examples of men who do bad work that one can expect an improvement to take place in the industry in this country. Works executed abroad, as a rule, are carried out by men who seem to take a great interest in their work. Even the ordinary labourer knows what he has to do and does it, whereas in this country there are so many instances of scamping, not only in reinforced work but also in other branches of structural engineering, that it is only strict supervision in the past that practically made our work some of the best in the world, though I have very clear remembrances before me of seeing wooden rivets and other such things being done, and I daresay there are many in this room who have seen likewise. We do not hear of wood being put in for steel in reinforced concrete—that is to say, by the designer—but many accidents have proved that wood has been put in and has been the cause of the mischief.

ELECTROLYSIS.

A point that was investigated by the Science Committee some time ago was one which it is advisable, I think, for me to call attention to, and that is, electrolysis. I have watched one work where this mischief occurred, and only about a couple of months ago I made a further examination and found that the mischief was still increasing. It is a moot point with some as to whether there is any electrolytic action or not, but personally I have very little doubt upon the subject, so that it behoves all designers and contractors to see that there is no possible chance of electric currents finding their way into steel reinforcing, because, should they do so, I do not think there is any doubt whatever that it will mean the eventual disruption of the concrete. It only requires the exercise of care to put a stop to this and to be perfectly certain that all cables throughout the reinforced concrete work are properly insulated, and that there are no stray currents wandering about the work.

CALCULATIONS.

I think a good many of my hearers will agree with me in regard to the calculations of reinforced concrete work that a great deal too much mathematics has been imported into the subject, and that common sense has had to take a back seat.

If we were dealing with two materials, both of which were absolutely constant—that is to say, the concrete constant within a month after it was made in strength as the steel is immediately after it is rolled—then it would be possible to go in for mathematical formulæ of a high order; but where you have a material—*i.e.*, concrete—the strength of which, if all portions are good, is increasing day by day, and in some cases attaining a strength of three, four, or more times that which it was originally calculated for, how is it possible in these ways to formulate any formulæ which are even moderately correct? It is far better to use more common sense and simplify or formulate empirical rules which you know are absolutely safe in their application. It is no good trying to extract the square root of two—it is useless. I have seen cases where the stresses have been worked out to five places of decimals, and it could all have been done by mental arithmetic, and the result would have been so close that it was really not worth while troubling about.

I see cases constantly where rods are put into work varied in diameter to 32nds of an inch. That is not at all necessary. It wastes time on the work, and in a great many cases a wrong rod goes into the wrong place. It is far better to adhere in all cases to commercial sizes, never advancing beyond 16ths of an inch and if possible advancing only by $\frac{1}{8}$ths, as by that means the size on the road is clear to the naked eye and does not require a caliper to be put upon it to find out whether it is 21-32 or 22-32 of an inch. Such reinforcements as these simply show how designers lack common sense and bring the work into disrepute. Exactly the same thing takes place with calculations submitted for approval. A mass of figures will be carried out into millions where the whole lot can be simplified by reduction into tens or hundreds, and the elimination of anything beyond two places of decimals. The simpler calculations are made the less liability there is of errors creeping in, and if an error does creep in, then it is much easier to discover; but where a whole foolscap page of figures is used to arrive at a result, it means in a great many cases simply courting disaster, as well as being also a waste of time and expense with a view to possibly saving a pennyworth of steel.

I have always been a great enemy to the complicated calculations, and I do not think they are at all necessary. It only brings to mind the largest building that has ever been constructed in London. I am now speaking of as long ago as about five-and-twenty years. In an arch rib there was a deficiency in the centre of the span of, I think, half an inch in a large sectional area of steel. To make up this deficiency in compression a bar 2 in. by $\frac{1}{4}$ in. was riveted on, and this only for a length of two or three feet, thus making mathematics simply an absurdity, whereas no common-sense engineer would for the sake of half an inch in one hundred square inches ever think of doing anything so absurd.

The same applies to reinforced concrete. It is far better to work in all cases to commercial sizes of rods, even if there be a slight deficiency in sectional area, than to put in multiples of rods of all diameter so as to make up a given sectional area. This entails an enormous amount of work not only upon the designer, but also upon the foreman who has to take charge of the work, and upon everybody connected therewith, and it is of no practical value whatever. Therefore I should like to see in all rules made for reinforced concrete that common sense should enter more largely into the formulæ that are given to the world, and not a mass of mathematics provided, which the authors know perfectly well are even then empirical, because one material is a constant and the other is an inconstant.

FAILURES.

The Committees of the Concrete Institute have been seriously considering for a long time past the question of failures, and there is no doubt that if one could get at the whole facts of the cases with regard to failures they would be most instructive. As a rule, every work that fails has been kept in the background, for fear that the contractor or somebody in connection therewith would suffer either from the commercial point of view or the professional. It will be found, I think, as a rule, that the causes of failure are ones that can easily be allocated with the exercise of a little common sense and supervision. It is, as a rule, due to great carelessness on the part of foremen and workmen employed that the failures take place. Therefore it is advisable in all cases that the principal causes of failures should be known, in order that specifications may be made so rigid that there is little likelihood of these mishaps. It is the fear of failure that debars to a certain extent reinforced concrete from being used by many clients, and therefore it should be the object of all concerned with this matter, from a professional as well as from a commercial standpoint, to see that such care be exercised in the whole of the work, both as to designing, checking, and construction, that no one, for one single instant, should have any doubt but that reinforced concrete is for certain classes of work the best that can be employed.

It is no good, for instance, if a failure takes place where it can be fairly traced to a bad aggregate to keep the same a secret, because it means that others possibly in the same neighbourhood go on constructing work therewith and with the possible chance that failure will also occur, whereas had the fact of the failure been generally known there is not much likelihood that a second disaster in the same locality would occur from the same cause. So that I make a confident appeal to all here to-night, and to all who may read the address, that wherever a failure in reinforced concrete takes place they will at once communicate with the Concrete Institute and let them have the facts of the case, not necessarily for publication, but to enable the Institute to so word their suggested specification as to guard against a possible accident taking place in the future.

It is only by failure that we are enlightened. All through life it is the same way, and we only learn and remember our alphabet at school by making mistakes and being punished for such mistakes so that we understand. So it is in a minor degree with the subject I am speaking to you about to-night; the elucidation of a failure may be the means in the long run of saving an enormous amount of money.

CONCLUSION.

I think I have trespassed on your time more than I intended, but I trust that the matter of this address is one that we all have at heart; and if it is the means of causing work generally throughout the country to be carried out in a better manner than a lot has been done in the past, then it will be of some avail

THE VOTE OF THANKS.

Mr. H. Percy Boulnois, M.Inst.C.E., ***Chairman of Council Royal Sanitary Institute, Vice-President Concrete Institute,*** *after proposing a vote of thanks to the President for his address and some general remarks regarding same, said, as far as the remarks on Portland cement were concerned:* "I am one of the old school, who still think it necessary to weather cement." *He went on to say:* "We are told by the President that we do more harm than good by weathering it, but the present method of making cement, the grinding and the hydrating and so on that goes on, makes cement that can be used immediately it comes from the kiln. In the old days we called it hot cement, which had to be cooled or ærated."

Regarding aggregates and moisture he was of the same opinion as the President "that excessive ramming is no doubt very often overdone."

"Then, the President's remarks on the effects of cold are exceedingly interesting. I have always suspected, though I have never carried out any experiments myself, that certainly frost had a very evil effect upon all classes of concrete. Then with regard to his testing of floors, there again he gives us some valuable hints.

"Regarding failures, he says they are very often due to shuttering and to strutting. That is, we know, very often the case. People are very often in a great hurry to remove their centering with very bad results, and his advice on that point is excellent.

"Then, with regard to the education of the clerks of works and of men, there, I quite agree, good work can be done by the London County Council if they will continue the education not only of the clerks of works, but also of the men who are engaged specially on this reinforced concrete work. We all know, in dealing with that class of work, how absolutely important it is that every man should be doing his best in putting that concrete in, and should not shirk, otherwise disaster may follow. We can standardise materials, but unfortunately we cannot standardise men, and it is the personal element of the man that comes into our works so enormously, and the best design may be ruined by bad workmanship. That none of us can escape from. Then, with regard to electrolysis, I think the fear of it has been rather overdone. I think it is evident that, with decent care, ordinary care, that could easily be prevented; and I agree with the President that the question, whether it does affect reinforced concrete to the extent that people imagine, is a little doubtful.

"Now, I also agree with him about over-calculation. Really it is very often far too elaborate, and I absolutely and entirely endorse all that he says with regard to that. It really frightens one sometimes to go into the question of reinforced concrete, because of these enormous calculations that have to be made, as, he says, to the fifth place of decimals. It really brings reinforced concrete almost into discredit.

"Last of all, he speaks of the importance of failures. As a man who has been an engineer for some fifty years or more, I entirely agree with him. I personally have learned more by my failures than I ever did by my successes, and young men should take that to heart and not be ashamed when they make a failure. Of course, we rub it into ourselves when we do make a failure, and consequently we do not forget and we learn a great deal more by failure than we do by success. And in this class of work, particularly with which this Institute deals, we shall learn a great deal if members and others will let us know when a failure takes place, in order that we may make some investigations, and those investigations properly followed up will be of the greatest value to those who follow us in constructing work of that design."

Mr. C. S. Meik, M.Inst.C.E., ***Vice-President of the Concrete Institute.*** *after seconding the vote of thanks to the President, said:* "I also am quite at one with the President's remarks about the workmen. The old-fashioned method of making concrete is, of course, quite out of place when you are dealing with material like reinforced concrete, and once you get a labourer who has been used to the old style, you cannot get him to make good work in the new style.

"I also quite endorse all that the President has said about the necessity of making test cubes when you are doing important work.

"Now, as to the calculations, I quite agree with Mr. Boulnois and the President as to the necessity of simplifying calculations for this material as much as possible. What is the use of going to the fifth space of decimals and doing everything to a nicety when you know, as I do from experience and tests, that the concrete itself varies as much as 100 per cent. in the same work very often in the same day, due to different weather conditions or due to different workmen. Intricate calculations in a case like that are quite unnecessary and useless.

"Now, the President interpolated into his address a short statement about the test made by the Institution of Civil Engineers. I happen to be on the Committee that made these tests, and I hope that the results will be published shortly. They have been delayed from unavoidable circumstances, but within the next few months I think they will be issued. The machine, if I may call it so, for making these tests is still there, and should anybody want to make experiments the machine is at their service. The supervision, of course, of the tests will have to be carried out under the superintendence of some members or somebody from the Institution of Civil Engineers."

Mr. E. Fiander Etchells, F.Phys.Soc., *after supporting the vote of thanks, went on to say that* "as regards standard notation, the Institute may certainly congratulate itself on the fact that its notation has been adopted, both in schools and in books, and by official authorities and by other technical institutions, to the exclusion of all counter-proposals, and if the same success follows the application of these principles to the formulæ of structural engineering generally, then this Institute will have done something to benefit the next generation.

"The President has also spoken about a Committee which was to deal with the widening of the scope of this Institute, and he refers to it as the Improvements Committee. Well, as Chairman of that Improvements Committee, I welcome any suggestions which the members have to make for the betterment of the Institute and the solidification of the profession.

"The President has also spoken about the title. He says that the sub-title is an institution for structural engineers and architects. I would like it to be more than that; I would like it to be truer to the facts, and make it the institution for structural engineers and architects, because it is the institution *par excellence* for them.

"And then I would also suggest that in doing that, the David of the Concrete Institute is not in any way challenging the Goliath of the Institution of Civil Engineers, for the Civil Engineers have a far wider aim than structural engineering only; they claim practically the whole of civil engineering, and the structural engineering is just a branch. Perhaps it may be in a rather distant future that the Civil Engineers themselves will become an institution of engineers, as they would at once if they included military engineers, who at present are only accepted as associates.

"I should like—and strongly would I desire it, too—that the Civil Engineers should cover the whole profession and ultimately include the Royal Engineers. Then we should have engineering a profession as deep and as profound, and as wide and as dignified, as the so-called learned professions of the Army, the Law and the Church.

"With regard to another point raised by our very practical President, of Fletton bricks, I would like to utter a word of warning as to other bricks besides Flettons—any bricks containing sulphur compounds and any bricks containing pyrites. In the case of the Flettons the trouble is with a fossil known as the ammonitea. A curly little thing it is, and it is mostly full of sulphur, but it is principally in the case of the Oxford clays where the ammonitea is a dangerous animal, even after he is dead. When he was alive he was mostly composed of calcium carbonates and silica compounds, but long after his death the house that he has vacated has become filled with pyrites, which have been solidified or settled down from the enfiltrations of water, and I have seen even in books that bricklayers are to beware of clay containing ammonitea. It is not the ammonitea which is the danger, it is the sulphur which fills his house after he is dead.

"As to the remarks concerning stress on a concrete beam under a load, this depends upon the span and the depth of the beam, and the amount of the steel, and the condition of fixing of the ends, but not upon the strength of the concrete. The stress is an external fact, but the deflection does vary inversely with the elastic modulus, and the elastic modulus of concrete increases with its age. If the concrete is new the elastic modulus is rather low and the deflection is high.

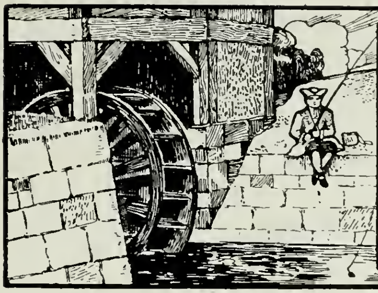
"The President has also spoken about intricate calculations, and he has spoken about their introduction into various documents. Well then, the Building Authority must be in a very difficult position. In the first draft of the Regulations which was issued to the technical societies the formulæ were so very simple, but certain technical societies demurred and stated that the formulæ were empirical, approximate, and inaccurate. The Building Authority, anxious to please, made them truer and scientific. But still the other side are not pleased even now. However, I personally agree with very simple formulæ.

"I would suggest that this Institute might have a Formulæ Committee, which would discuss and arrange simple formulæ which would be approximations to any official Regulations or official Reports which there may be."

After some remarks regarding decimals, *Mr. Etchells said regarding failures that* "you should not judge reinforced concrete by the failures which have happened. You do not judge automobile engineering by the accidents which happened in the early days when the cars were built higher and they overturned on going round corners. You do not judge aviation by the failures of aeroplanes. So do not judge reinforced concrete by practically the accidents which happened in the days of youthful indiscretions. It is a material which can be dealt with by the ordinary laws of mechanics, supplemented by common sense."

Mr. Boulnois then put the motion, which was carried by acclamation.

After some words of thanks from the President, the meeting terminated.



ELEVATED REINFORCED CONCRETE LIME-WATER TANKS for the FRIMLEY & FARNBOROUGH DISTRICT WATER COMPANY.

By CLAUDE PAIN.

The following particulars should be of interest to our readers, as the article once more proves that reinforced concrete not only lends itself excellently for work of this kind but is considerably cheaper than cast iron.—ED.

The tanks described in this article were constructed in connection with a water softening plant for the Frimley and Farnborough District Water Company at their Greywell Pumping Station, about six miles from Basingstoke.

The softening plant, which is worked on Clark's process and has a maximum capacity sufficient to reduce the hardness of 1,000,000 gallons per day from 18 to 6 degrees of hardness, requires the use of some 120,000 gallons of lime water per day, and it is for the mixing and storing of the lime water that these tanks are required. The total capacity of the tanks was made 147,000 gallons, which is 22½ per cent. over the actual quantity required, to allow for the accumulation of spent lime deposit in the bottom of the tanks before washing out was necessary; and, for general convenience of working, they were made in two independent units of approximately equal capacity.

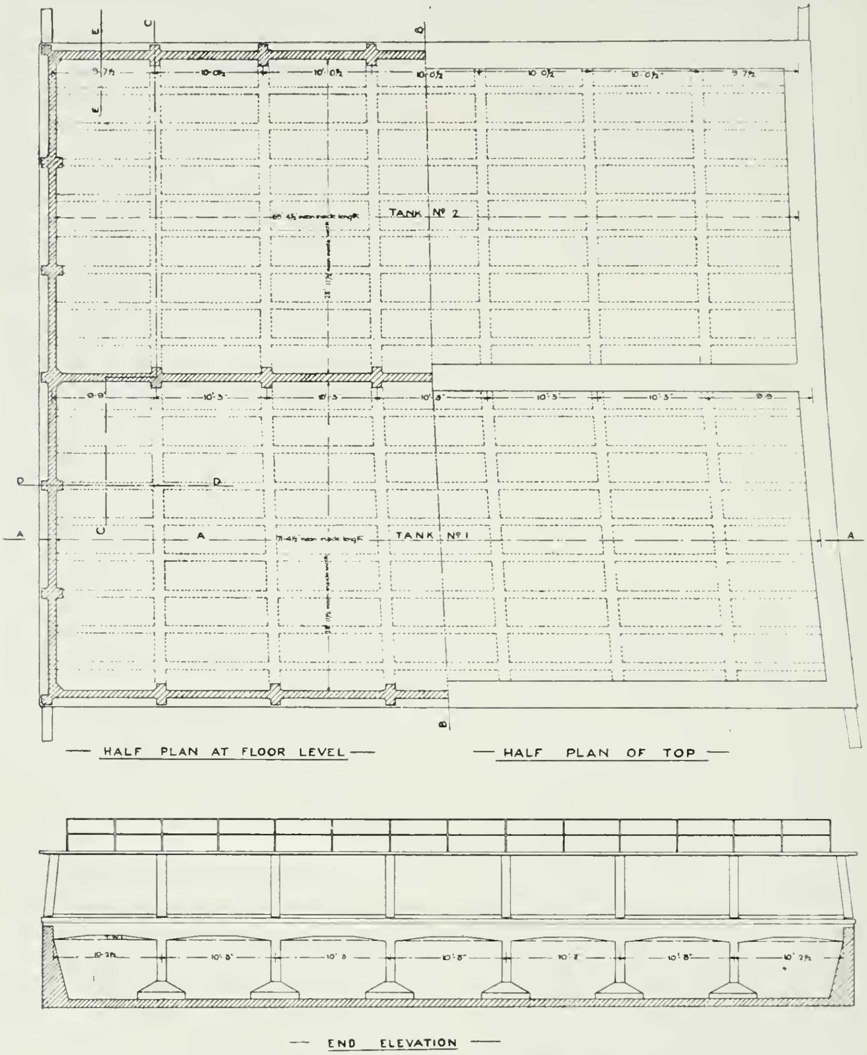
Showing Method of Reinforcement.
ELEVATED REINFORCED CONCRETE LIME-WATER TANKS.

As will be seen from the illustrations, the tanks are built side by side over subsiding basins, in which the actual softening process is carried out, and are rectangular in section, each tank being approximately 70 ft. long by 30 ft. wide by 6 ft. 3 in. deep, with a maximum depth of water of 5 ft. 9 in.

The subsiding basins were built of 6 to 1 mass concrete, but the lime-water tanks were constructed throughout of reinforced concrete in the proportions by volume of 4 parts of stone and 2 of sand to 1 of cement. The stone was flint gravel screened to pass through a ¾-in.

mesh and afterwards washed to rid it of all dirt. The proportion of sand named was gauged from the volume of water needed to fill the voids in the gravel, with an addition of 10 per cent.

Twisted steel bars were used throughout for the reinforcement, it being considered essential that maximum adhesion should be obtained in resisting the temperature effects to which the tanks would be subjected in their exposed position. The bars were specified to have, when twisted, an elastic limit of not less than 56,000 lb. per square inch with a minimum elongation of 10 per cent. in 8 in. and to stand the usual bending test.



Elevated Reinforced Concrete Lime-Water Tanks for the Frimley and Farnborough District Water Co.

The general design will be seen to consist of the beam and slab type, which was chosen as being the most suitable.

The calculations allowed for a unit compressive stress in the concrete of 600 lb. per sq. in., no tension being allowed, and a unit tensile stress in the steel of 20,000 lb. per sq. in., the ratio of the moduli of elasticity being taken at 15 and the stress strain curve as straight. One of the chief considerations in the design, owing to both floors and walls being exposed, was the provision of adequate reinforcement to avoid temperature cracks, such as invariably develop in plain concrete, especially in view of the tops of the tanks being unrestrained by outside forces, whereas the bottom of the end and division walls at any rate are restrained from contracting and expanding by their fixture to the walls of the subsiding basins beneath. It was realised that diagonal stresses would be set up in the walls and floors from this cause, especially at the corners, but it was considered that the web strength of the wall and floor slabs would be sufficient to provide for these stresses without diagonal reinforcement, and except in the case of one corner of the floor where the temperature range is greatest and a slight hair crack developed in the early stages of construction this anticipation has been realised. Where it was thought desirable to provide reinforcement for direct temperature stresses a steel ratio of 0·24 per cent. was allowed to compensate for a possible temperature range of 50°.

The columns, which rise from bases 4 ft. 6 in. square laid directly on the concrete bottom of the subsiding basins, are set at approximately 10-ft. centres in both directions, and are 3 ft. 10 in. high and 10 in. square, reinforced with four $\frac{5}{8}$-in. bars tied every 8 in. with $\frac{1}{4}$-in bars.

The primary beams, which are at 10-ft. centres, are 24 in. deep by 11 in. wide over the columns, reinforced by four $\frac{5}{8}$-in. bars at the top and two $\frac{1}{2}$-in. bars at the bottom, and are 20 in. deep by 11 in. wide in the middle, reinforced by two $\frac{5}{8}$-in. bars at the top and two $\frac{5}{8}$-in. and two $\frac{1}{2}$-in. bars at the bottom, hangers of $\frac{3}{16}$-in. diameter wire being provided at 11-in. centres up to the third points of the span.

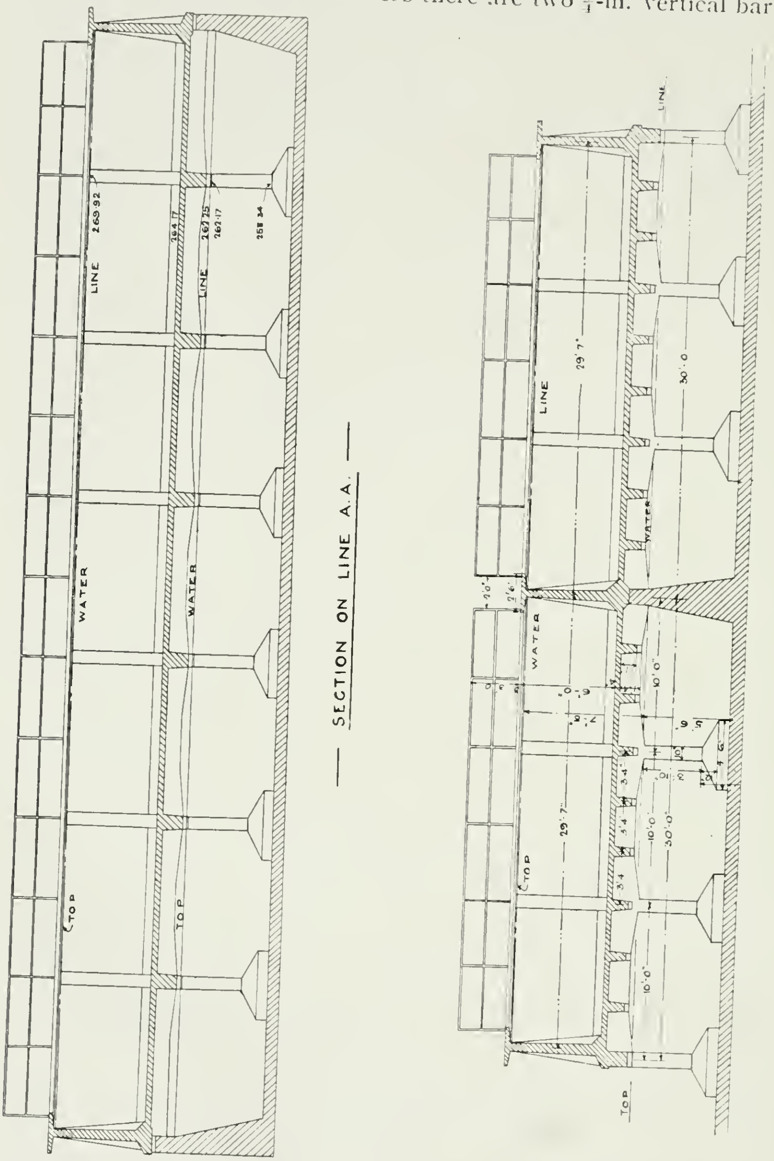
The secondary beams, which are at 3-ft. 4-in. centres, are 20 in. deep by 7 in. wide at the ends, reinforced by three $\frac{3}{8}$-in. bars at the top and two $\frac{3}{8}$-in. bars at the bottom, and are 17 in. deep by 7 in. wide in the middle, reinforced by four $\frac{3}{8}$-in. bars at the bottom, hangers of $\frac{3}{16}$-in. diameter wire being provided at 8-in. centres up to 17 in. from the middle of the span.

The floor slab, which is $4\frac{1}{2}$ in. thick, is reinforced by $\frac{1}{4}$-in. bars at 6-in. centres, alternate bars being bent up at the third points of the spans to provide for negative bending over the secondary beams, distributing bars $\frac{1}{4}$ in. diameter being provided at 13-in. centres.

The walls consist of buttresses at 10-ft. centres, 10 in. wide and tapering in thickness from 24 in. at the root to 6 in. at the top, with panels between tapering from 9 in. thick at the bottom to 6 in. at the top.

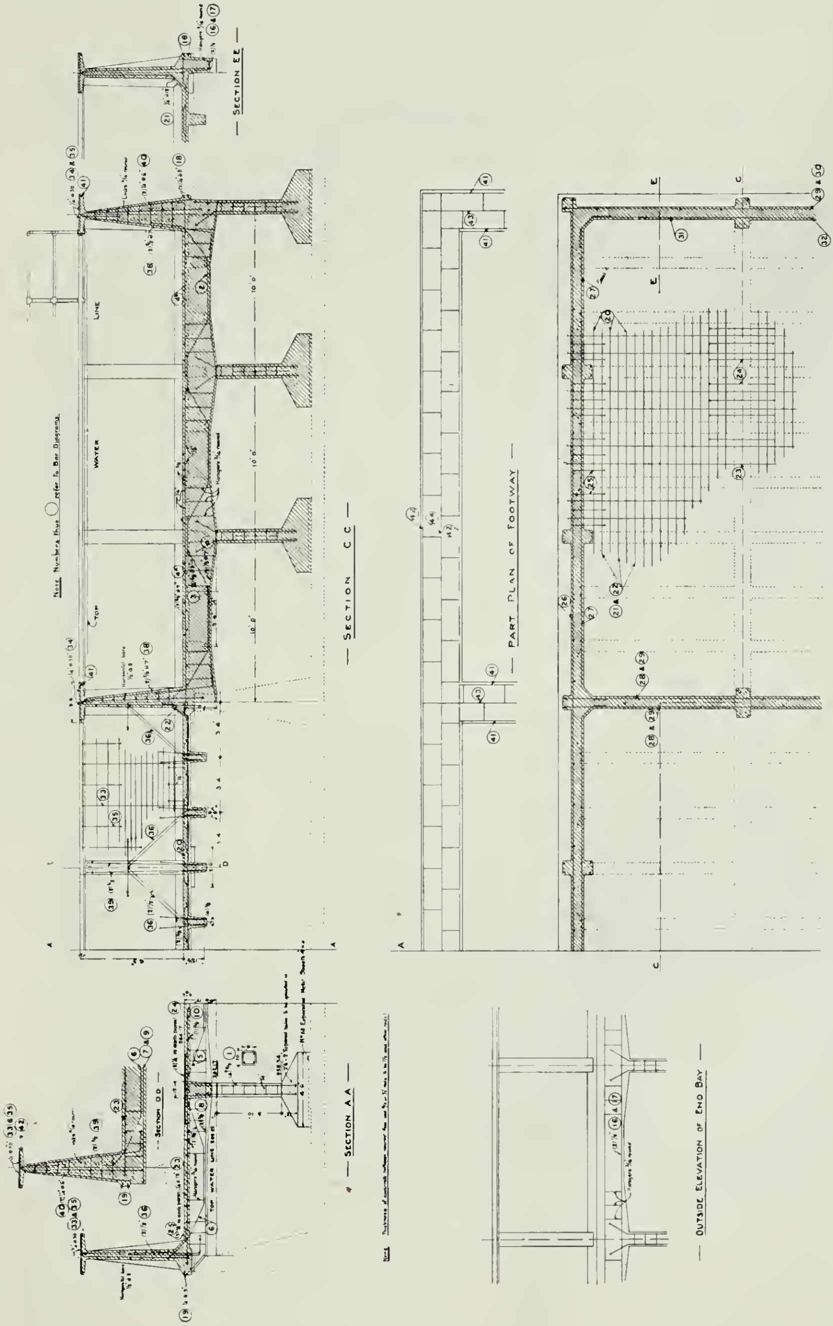
The buttresses in the side and division walls are anchored at the root to the ends of the primary beams by a reinforcement of two $\frac{5}{8}$-in. bars and the upturned ends of the lower pair of $\frac{5}{8}$-in. bars in the primary beams. The torque from the ends of the secondary beams being insufficient to balance the reaction at the root of the buttresses in the end walls, sufficient gravity reaction had to be obtained by carrying these buttresses on saddles and bringing the weight of the adjacent secondary beams to the buttresses by two $\frac{1}{2}$-in. diagonal suspension bars instead of supporting their ends on the wall of the basin beneath, and in this case there are only two ⅜-in. bars for the anchorage at the root of each buttress into the end of the secondary beams.

On the outside of each buttress there are two ¼-in. vertical bars to bond the face against temperature cracks, and horizontal links of $\frac{3}{16}$-in. round wire tie the vertical rods together at intervals of 8 in.



Elevated Reinforced Concrete Lime-Water Tanks for the Frimley and Farnborough District Water Co.

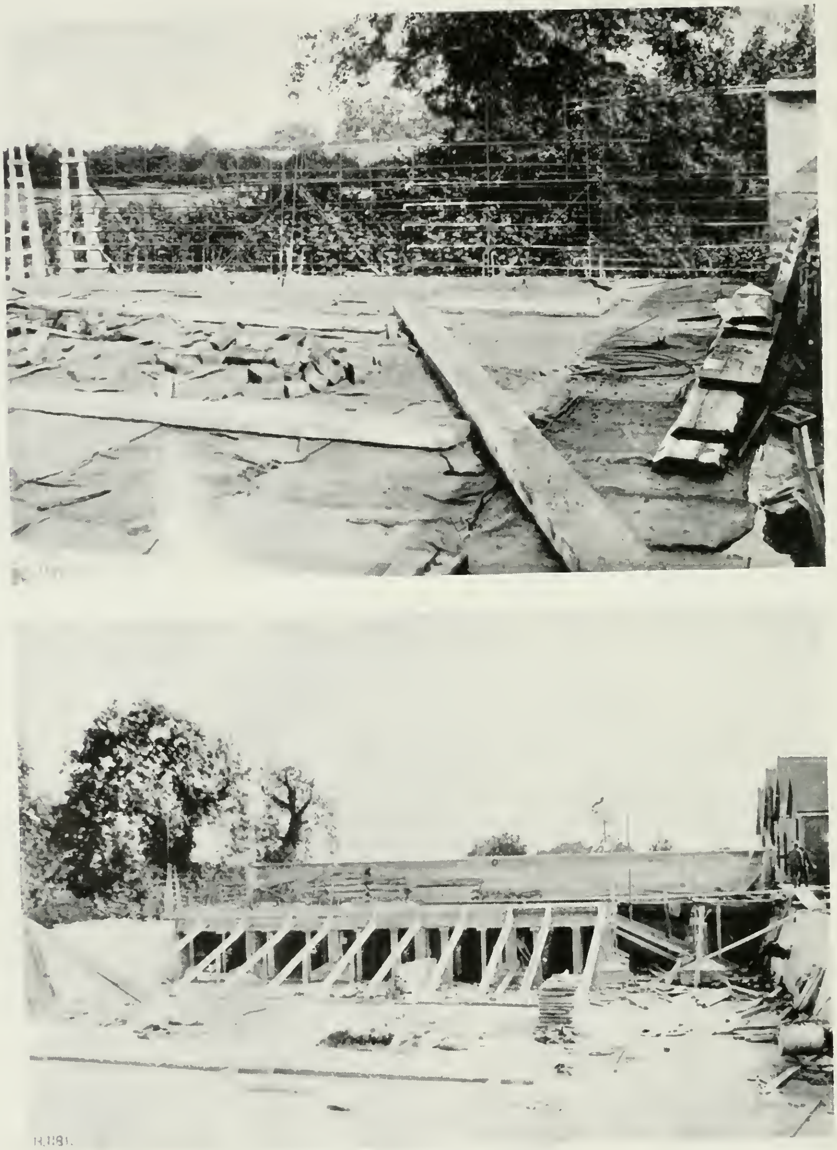
The wall panels between the buttresses are reinforced horizontally with ½-in. bars at 8-in. centres for the whole height on the outside face and for two-thirds of the height on the inside face, $\frac{1}{4}$-in. vertical distributing bars being provided at 30-in. centres.



Elevated Reinforced Concrete Lime-Water Tanks for the Frimley and Farnborough District Water Co.

The weight of the panels in the side walls is carried by fascia beams and in the end walls by the diagonal suspension bars already mentioned.

Footways are provided around the tanks on the top of all walls, so that in frosty weather the ice can be broken. The slab forming the footway is 2 ft. 6 in. wide and $3\frac{1}{2}$ in. thick, reinforced longitudinally by three $\frac{3}{8}$-in. bars and transversely by extensions of the $\frac{1}{4}$-in. vertical distributing bars in the wall panels.



Showing Tanks in course of construction.
ELEVATED REINFORCED CONCRETE LIME-WATER TANKS.

One tank was constructed at a time, and the floor of each tank was laid in three sections, the division between each section being made along the middle of a span. These divisions in the first tank were washed over with neat cement grout as the concrete of the next section was put in place, but in the second tank mortar was used instead of the grout, as it was found that the new concrete drew away from the old in setting.

The inside face of each tank was rendered with 2 to 1 cement mortar, and as soon as this was set and the pipework completed the first tank was tested and put into regular use. Afterwards the outside faces were rendered to obliterate the inequalities of the surface left by the warping of the shuttering caused by the exceptionally dry weather of the summer of 1911, during which the tanks were constructed.

No measurable deflections could be noted with a micrometer in the various members, and beyond a slight weeping along some of the division lines between the sections the tanks have been found to be water-tight.

Probably no shrinkage would have been experienced at the joints that could not have been taken up by the elasticity of the rendering had it not been for the dry weather. No anxiety is felt, however, as to the corrosion of the steel, as the water contained in the tanks is heavily charged with lime, which, it is anticipated, will eventually seal up the joints.

The cost of the reinforced concrete work amounted to 1·2 penny per gallon of capacity, which is roughly half the cost of cast-iron tanks on rolled steel joists and stancheons.

The designs were prepared by Mr. Claude Pain for Mr. Robert Nunn, the Engineer to the Frimley and Farnborough District Water Co., and the contractors for the work were Messrs. A. H. Ball & Co., of Farnham.

STANDARD ALGEBRAICAL NOTATION.

By E. FIANDER ETCHELLS, F.Phys.Soc., A.M.I.Mech.E.
Vice-Chairman of the Science Committee of the Concrete Institute.

The question of an International Notation is one of great importance for the better understanding of investigations and regulations relating to reinforced concrete. The initiative for obtaining an International Notation in respect of reinforced concrete came, it is curious to say, from England, a late comer in the field of reinforced concrete practice. The first Chairman of the Concrete Institute, Mr. Edwin O. Sachs, F.R.S.Ed., pressed this point actively at the International Association for the Testing of Materials and on the International Reinforced Concrete Commission. It was further actively taken up by the Concrete Institute, thanks largely to the energy of Mr. E. Fiander Etchells, F.Phys.Soc., A.M.I.Mech.E. Unfortunately the suggestion for internationalisation came somewhat too late in the day to obtain absolute concord among all nations. Nevertheless, a considerable amount of unanimity has been arrived at on certain primary points, and the result has been that if no uniform notation has been obtained for the whole of the civilised world, at least certain guiding principles have been arrived at, and the notation may broadly be divided into two sections for the English-speaking countries and the continental countries. England, being such a late comer into the reinforced concrete arena, has been at a disadvantage in pressing the matter, even among the English-speaking countries, for the United States were obviously far advanced in reinforced concrete work before the matter was seriously brought up. But, for all that, considerable progress has been made. As to England herself, this work has been of the highest possible benefit, as indicated in the article given below, and quite a number of influential bodies are adopting a uniform form of notation.—ED.

THE Editor has asked for an article dealing with standard notation, giving a short history of the work done in this matter, and showing how the final notation was arrived at.

This is rather a comprehensive request, for this question of standard notation was under consideration in Egyptian temples more than 3,500 years ago, and the final report of the Concrete Institute has not been drafted yet. However, it is proposed to shorten the history by omitting all reference to work earlier than 1884, when Professor Jamieson wrote a paper for the Institution of Electrical Engineers on "Electrical Definitions Nomenclature and Notation."

On November 5th, 1884, Monsieur Hospitalier made a communication to the Société Internationale des Electriciens on "The Standardisation of Electrical Notations" (Paris 1884).

Some of the points of the Hospitalier Notation were adopted by the Concrete Institute in 1909, and may be found in their Transactions. (See Vol. I., part 3, page XL. of the Notes.)

Professor Jamieson's paper was read on May 14th, 1885. A committee of the Institution of Electrical Engineers was afterwards formed, but they dealt principally with the definitions and nomenclature.

There were also conferences on electrical notation in America in 1893, and again in 1905.

The next step appears to have been taken by the Civil and Mechanical

Engineers' Society. On January 2nd, 1908, the above society held a conference on Standard Notation for Engineering Formulæ.

A number of papers were presented, including one by the writer, who laid down nine guiding principles, all of which have since formed the basis of later work by other technical societies.*

These principles represented an effort to establish a natural system of notation which should find its sanction in Psychology and Logic, instead of following the diverse and irrelevant symbols arbitrarily introduced by various authors.

It was proposed to eliminate the personal element from notations, and to leave the symbols to be a practical and effective index to the quantities represented.

On July 29th and August 5th, 1909, it was suggested that the Concrete Institute should take up this question of Standard Notation.†

On October 29th, 1908, the Council of the Concrete Institute referred these recommendations to their Science Committee.

This Committee, at its first meeting on December 3rd, 1908, discussed these suggestions, and at the same time had under consideration the principles formulated at the Conference of the Civil and Mechanical Engineers' Society; and also considered a copy of an international notation which had been put forward at Basle on October 12th, 1908, by the International Commission on Reinforced Concrete (formed by the parent body, the International Association for Testing Materials).

That Commission, by-the-by, which comprised some 19 members representing nine countries under the chairmanship of Professor Schüle, of Zürich, left the detail of obtaining an agreement on the subject of uniform notation to a special sub-committee comprising Professor Melan, of Prague; M. Mesnager, of Paris; Professor Maillart, of Zürich; Meinheer Rutgers, of Rotterdam; and Mr. Edwin O. Sachs, of London, who in their turn recommended that, (a) small roman letters should be employed for longitudinal data and loads per unit length; (*b*) roman capitals should be used for areas and forces; (*c*) small Greek letters should be used for coefficients and for expressing stresses (forces per unit area), and further prepared a schedule of notations based on these principles.‡

It should perhaps immediately be added here that this particular recommendation was adopted at the Copenhagen meeting of the Commission of September 5th, 1909, and further confirmed at the New York meeting of the International Association for the Testing of Materials in September, 1912, but that in both instances it was clearly put forward that the uniform notation suggested would have to have certain modifications for the English-speaking countries, and that some agreement should be arrived at in this respect by the United States of America, Great Britain and its Colonies, so that, as a matter of fact, a uniform notation would only be uniform in part with a sub-section for the English-speaking countries and a sub-section for the Continental countries.

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* *Vide* Transactions of the Civil and Mechanical Engineers' Society (now the Society of Engineers), 49th Session, 1907-8, page 71.

Also see Transactions of the Concrete Institute, Vol. I., Part 1, page xiv. of Notes.

Also see Report of the Royal Institute of British Architects on Reinforced Concrete, 1911 (Key to the Notation).

Also see Presidential Address of the Association of Engineers in Charge on October 13th, 1909.

† *Vide* Transactions of the Concrete Institute, Vol. I., Part 3, page xxxvi. of the Notes.

‡ CONCRETE AND CONSTRUCTIONAL ENGINEERING, page 660, September, 1912 (Vol. VII. No. 9).

On January 14th, 1909, the Science Committee, under the chairmanship of Mr. William Dunn, resolved to proceed with the notation in conformity with the principles formulated by the writer on January 2nd, 1908, which were partially supported by the recommendations of the International Commission (*i.e.*, in respect of the use of three alphabets and the partial use of initial letters).

With regard to a universal notation the Concrete Institute were of opinion that :—

> "An international system of notation would not appear suitable for daily national use, for in so far as it was self-explanatory to the English-speaking people, so far would it be a matter of memory to the other nations, and *vice versa*.
>
> "An artificial and arbitrary notation devised by savants for international use could not hope to obtain the unanimous support of any nation, for a polyglot notation would be more of a hindrance than a help to the majority in any country."

It was suggested that the notation should follow the language of the text—*i.e.*, that French notation should be an abbreviation of French words and German notation of German words.

It will thus be seen that an international notation should be a natural derivative of an international language (if any).

A special sub-committee* was appointed to draw up a list of symbols in conformity with the principles agreed upon and to prepare a draft Report. Each symbol was discussed separately, and each member of the committee gave careful attention to all the pros and cons of every suggestion. The draft Report was afterwards discussed by the full committee under the chairmanship of Mr. William Dunn, F.R.I.B.A., and was afterwards discussed and approved by the Council of the Concrete Institute.

The result was the document issued to the various technical societies and institutions and to the technical Press in January, 1910, entitled "Standard Algebraical Notation."

ADOPTION OF THE NOTATION.

Since that date the notation has been adopted by the British Joint Committee on Reinforced Concrete. (*Vide* Second Report issued by the Royal Institute of British Architects 1911.)

It has also been adopted in the draft regulations in respect of Reinforced Concrete.

It has been favourably received in America, and is at present under the consideration of the American Joint Committee on Reinforced Concrete.

It is also now employed in the majority of recent text books on reinforced concrete or allied subjects.

It is also being adopted in the various college courses on reinforced concrete throughout the country.

**Vide* Transactions of the Concrete Institute, Vol I., Part 3, page xxxvii. of the Notes. Also *vide* Transactions of the Concrete Institute, Vol. II., Part 1, page xiii.

FUTURE WORK.

The Concrete Institute are at present engaged in applying the principles to the symbols used in other branches of structural engineering. They will be glad to receive and consider any suggestions which may be made to them through the Secretary.

These suggestions may deal with broad questions of principle or even suggestions as to the uses of any particular symbol.

PREVENTION OF FUTURE CHAOS.

One of the great sources of confusion in notation is the translation of technical books by persons who may be able to translate the text, but who may not be competent to translate the equations ino more homely symbols. They read :—

E_b = module *d'élasticité béton*, but they seem afraid to translate this into—

E = *Elastic* modulus of the *concrete*, and so they only translate one side of the equation. They read :—

D = *Druck*kraft
Z = *Z*ugkraft

and yet they seem too timid to say—

C = Compression
T = Tension

and so we get—

D = Compression and
Z = Tension.

And this is handed out to us as a complete translation.

If readers would only take the trouble to write to all such "semi-translators," pointing out that they have only half done their work, it is probable that later editions would be more readable and more acceptable to the persons who require a translation.

Many a good book is spoiled by being only half translated. A complete translation necessitates more work for the translator; but "hard writing makes easy reading," and there are thousands of readers to one writer. Besides, it is presumed that the translator wishes his book to be widely read; therefore he should make it widely readable.

The latest form of the standard algebraical notation as applied to reinforced concrete formulæ is appended herewith.

KEY TO THE NOTATION.

(1) The notation is built up on the principle of an index.

(2) The significant words in any term are abbreviated down to their initial letter, and there are no exceptions.

(3) Capital letters indicate relative *greaterness*, including total forces, total loads, areas, volumes, moments, ratios, and constants.

(4) Small letters indicate relative *lesserness*, including intensity of forces, intensity of loads, and intensity of stresses, lineal dimensions (lengths, distances, &c.), ratios, and constants, &c.

(5) Dashed letters indicate ratios, such as $a_{\prime}$ $c_{\prime}$ $n_{\prime}$ &c., where the a, c, and n indicate the numerators in the respective ratios. The dash itself is mnemonic, and is an abbreviation of that longer dash which indicates division or ratio.

(6) Subscript letters are only used where one letter is insufficient; and the subscript letters themselves are the initial or distinctive letters of the qualifying words.

(7) Greek letters indicate ratios and constants. They are sparingly used, and are subject to the " initial letter " principle.

(8) A superscript dash indicates dimensions in feet, *vide* L.

STANDARD NOTATION.

(In beams) A = *Area* of tensile reinforcement (in square inches).
(In pillars) A = the effective *area* of the pillar.
A_c = net *area* of *concrete* in a pillar $= A - A_v$.
A_E = *Area equivalent* to some given area or *area* of an *equivalent section* or equivalent area.
A_L = cross-sectional *area* of one bar of the *lateral* reinforcement of a pillar (square inches).
A_s = cross-sectional *area* of a vertical or diagonal *shear* member, or group of shear members, in the length p, where p = pitch of stirrups.
A_v = *Area* of *vertical* reinforcement of a pillar in square inches.
a = *arm* of the resisting moment or lever *arm* (in inches).
$a_{,}$ = *arm* ratio $= a/d \therefore a_{,} d = a$.
B = *Bending* moment of the external loads and reactions (in pound inches).
B_1 B_2 = *Bending* moments at consecutive cross sections.
(generally) b = *breadth*.
(In tee beams) b = *breadth* of flange of beam (in inches).
b_r = *breadth* of *rib* of T beam (in inches).
β = *Buckling* factor in pillar equations.
C_1 C_2 C_3 = a series of *constants*.
(In beams) c = *compressive* stress on the compressed edge of the concrete (in pounds per square inch).
(In pillars) c = direct *compressive* stress on concrete not hooped.
c_m = *mean compressive* stress.
c_s = *compressive* stress on *steel*.
c_u = *compressive* stress on concrete at the *underside* of the slab (in tee beams).
$c_{,} = c/t$ = the ratio of c to t.
D = *Depth* in feet.
[The capital letter indicates the relative greaterness of the unit, but the symbol should only be used when it is necessary to distinguish between D and d. d' may be used for depth in feet [(if preferred.)]
(In circular sections generally) d = *diameter*.
(In rectangular sections generally) d = *depth*.
(In pillars) d = the *diameter* of the hooped core in inches.
(In beams) d = effective *depth* of the beam (in inches).
d_c = *depth* or *distance* of the *centre* of compression from the compressed edge of the beam.
d_n = *deflection*.
d_s = total *depth* of the *slab* (in inches).
d_v = *distance* between the centres of *vertical* bars in pillars measured perpendicular to the neutral axis.
$d_{,} = d/d_s$ = ratio of effective *depth* of beam to total *depth* of *slab*.
(generally) E = *Elastic* modulus of any material.
E_c = *Elastic* modulus of *concrete* (in pounds per square inch).
E_s = *Elastic* modulus of *steel* (in pounds per square inch).
e = *eccentricity* of the load measured from the centre of the pillar (in inches).

(In beams) f = *flexural* stress = extreme *fibre* stress, *i.e.* stress at the extreme "fibre" of any member under transverse load.

(In pillars) f = a *form* factor or constant which will vary according to whether the hooping is curvilinear or rectilinear, &c.

(generally) g = *gravity* coefficient = 32·2.

(In pillars) g = *gyration* radius.

I = *Inertia* moment of a member.

I_c = *Inertia* moment of *concrete* only.

I_s = *Inertia* moment of *steel* only.

I_{XX} = *Inertia* moment on axis XX when necessary.

I_{YY} = *Inertia* moment on axis YY when necessary.

i = *increased* stress permissible on the core of a pillar suitably hooped.

$i = c[1 + fsr]$.

L = *Length* in feet, or l' may be used [if preferred].

[The capital initial indicates the relative greaterness of the unit, but the symbol is only to be used when it is necessary to distinguish between L and l.]

l = *length* of a pillar or effective length of span of beam or slab.

m = *modular* ratio = E_s/E_c.

N = a *numerical* coefficient.

$N_2\ N_3\ N_4$ = a series of numerical coefficients.

n = *neutral* axis depth, *i.e.* depth of *neutral* axis from the extreme compressed edge (in inches).

$n_1 = n/d$ = the *neutral* axis ratio $\therefore n_1 d = n$.

P = total safe *pressure*.

(In pillars) p = the *pitch* or the laterals in inches (*i.e.* the axial spacing of the laterals).

(In shear formulæ) p = *pitch* or distance apart (centre to centre) of the shear members or groups of shear members (measured horizontally).

p_c = percentage of reinforcement.

$$\left[\frac{p_c}{100} = r \therefore p_c = 100r.\right]$$

π = *peripheral* ratio or the circumference of a circle to its diameter.

R = Resistance moments generally.

R_c = *Compressive Resistance* moment = *Resistance* moment of the beam in terms of the *compressive* stress (in pound inch units). (When necessary to distinguish from R_t.)

R_t = *Tensile Resistance* moment or Resistance moment in terms of the *tensile* stress (in pound inches). Only used when necessary to distinguish from R_c.

(In beams) $r = A_t/bd$ = *ratio* of area of tensile reinforcement to the area bd.

(In pillars) $r = V_h/V$ = the *ratio* of volumes, *i.e.* the ratio of the volume of *helical* or *horizontal* reinforcement to the volume of hooped core.

(In beams) S = the total shear in pounds at a vertical section.

S_F = Safety factor = $\frac{U}{W}$.

S_m = the *section modulus*.

(In pillars) s = *Spacing* factor or constant which will vary with the pitch of the laterals.

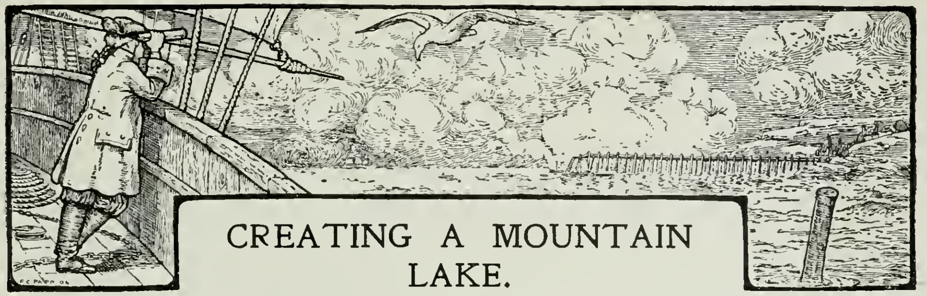
(In beams) s = intensity of the *shearing* stress on concrete in pounds per square inch.

s_s = *shearing* stress on the *steel* (in units of force per unit of area).

$s_1 = d_s/d$ = the *slab* depth ratio.

T = Total *tension* in the steel (in pounds).

T_1 T_2 = Total tensile forces at consecutive cross sections.
t = *tensile* stress on the steel (in pounds per square inch).
U = Total *ultimate* breaking load on any member.
[Compare W = *Working* load.]
u = intensity of *ultimate* crushing resistance of plain concrete per unit of area or *ultimate* compressive stress on prisms of concrete not hooped.
(generally) V = *Volume.*
(In pillars) V = *Volume* of hooped core in cubic inches.
V_h = *Volume* of *helical* reinforcement in cubic inches.
W = total *working* load or *weight* on any member.
W_F = the *working factor* = the reciprocal of the safety factor.
w = *weight* or *working* load per unit of length of span.



CREATING A MOUNTAIN LAKE.

THE HUGE MASONRY OF THE AUSTRALIAN BURRINJUCK DAM.

By F. M. CUTLACK.

Below we present some interesting particulars regarding the great Dam across the Gorge at Barren Jack Mountain in New South Wales.—ED.

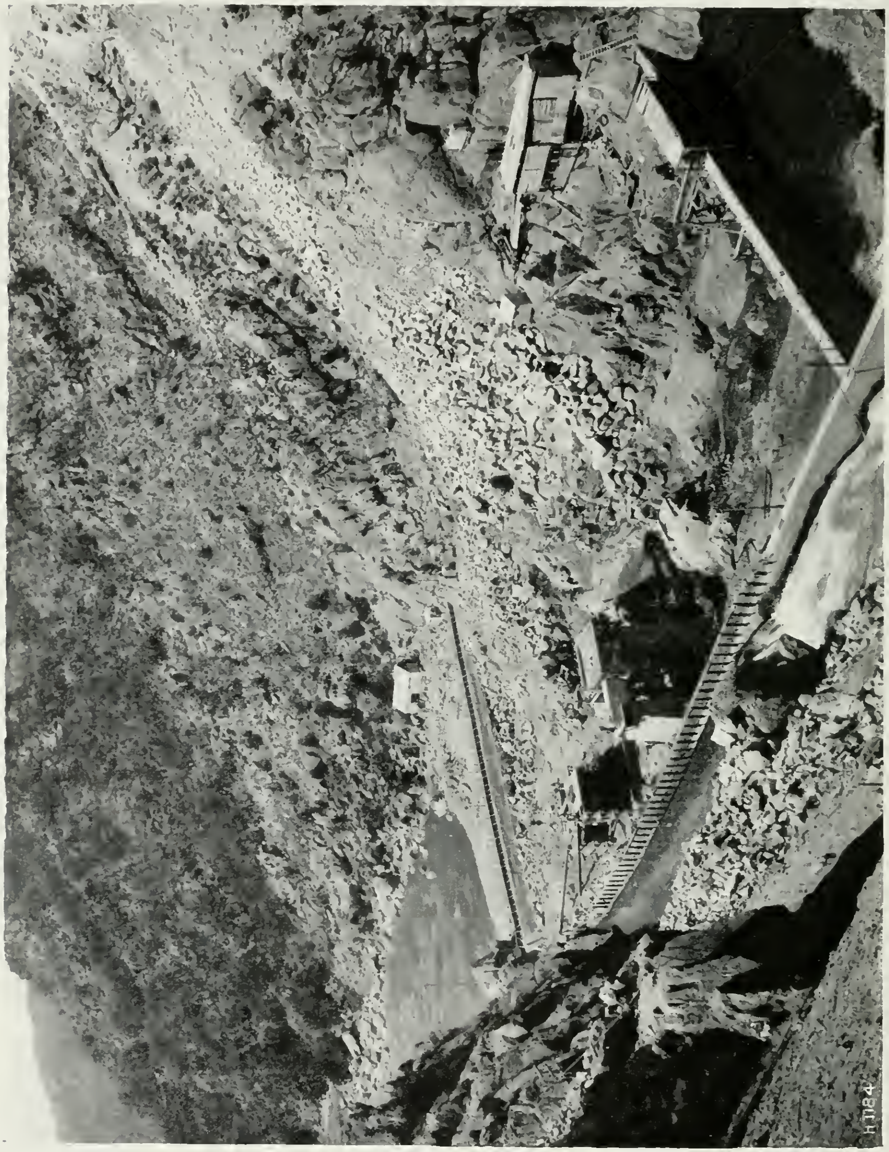
The great dam across the gorge at Barren Jack Mountain on the Murrumbidgee River in New South Wales is the second largest in the world. Only Assuan, in Egypt, eclipses it in size and power, but the Assuan Dam, though larger, cannot compare in picturesqueness with this big irrigation work in Australia.

The Burrinjuck Dam is a huge barrier across the deep river bed high up in the mountains at the river's source, and the ridges behind it, which will form the natural containing walls of the inland sea to be thus created, are nowhere lower than 2,000 ft. above the river bed level.

The dam will store a lake of water of 12,740 acres, sufficient to cover 766,324 acres to a depth of one foot. The waters stored and regulated by the dam flow down the channel of the river for 200 miles to Berembed, where the diversion weir and head works are situated. At Berembed a flow of 2,000 cu. ft. of water per second is available for irrigation purposes, without in any way interfering with the riparian rights along the whole length of the river channel. The main irrigation canal is 64 ft. wide at the water level, with a flow of 8 ft., equal to a discharge of 1,000 ft. per second, though this capacity will eventually be doubled.

As described by the constructing engineers, the dam is of concrete formed of broken granite ballast, silicious sand, and Portland cement, with displacer blocks embedded in the mass. The retaining wall is placed in the neck of the Barren Jack gorge, and is to be carried to an ultimate height of 240 ft. from foundation to crest, with an available maximum storage depth of 200 ft. from the invert of the outlet culvert to the full supply level of the dam. It is 784 ft. long and curved on plan to a radius of 1,200 ft., with the convex face upstream. The width of the base at the deepest part is 160 ft. The lower 20 ft. has vertical faces both up and down stream, and above these the width suddenly diminishes to 145 ft. Thence the upstream face has a batir of 1 in 20, and the back or downstream face a slope of 1 horizontal to $1\frac{1}{2}$ vertical up to a point 60 ft. below the crest. Above this the face will be continued vertically, and the back by a curve of about 108 ft. radius, convex downstream and terminating normally to the crest. The crest will be 18 ft. wide.

The supply outlet is by a tunnel set in the body of the main wall and measuring 14 ft. 3 in. by 13 ft. The outlet is to be controlled by suitable valves; worked by gearing from a masonry tower standing out upstream from the body of the main structure, and the flow regulated by a second set of valves near the downstream face. An access tunnel, 6 ft. by 4 ft., chiefly to facilitate inspection, will traverse the length of the dam 38 ft. below the crest and be approached by a shaft at each end. To control the action of water passing from the reservoir through the by-washes in time of flood a low weir will be made to form a pool or cushion for the descending waters, so that they may become quiescent before passing down the river channel. This is to prevent any cutting back action of the water which might eventually endanger the safety of the dam.



THE BURRINJUCK DAM, NEW SOUTH WALES, AUSTRALIA

Showing a Granite Foundation.

The foundations of the dam are as good as they can possibly be. They are built into a solid granite rock which forms the bed of the river just at this gorge. The structure is of cyclopean rubble, the material for the concrete and large blocks of stone being obtained from the granite from the sides of the gorge, which is extremely hard and very suitable for the purpose. So convenient is the material to hand that the cost per acre foot stored in completed dam will not be more than £1. This compares very favourably with the acre foot cost of the Assuan Masonry Dam (£2·72), though the Roosevelt Cyclopean Rubble Dam in Arizona, which presents many features similar to Burrinjuck, was estimated to cost only £·48 per acre.

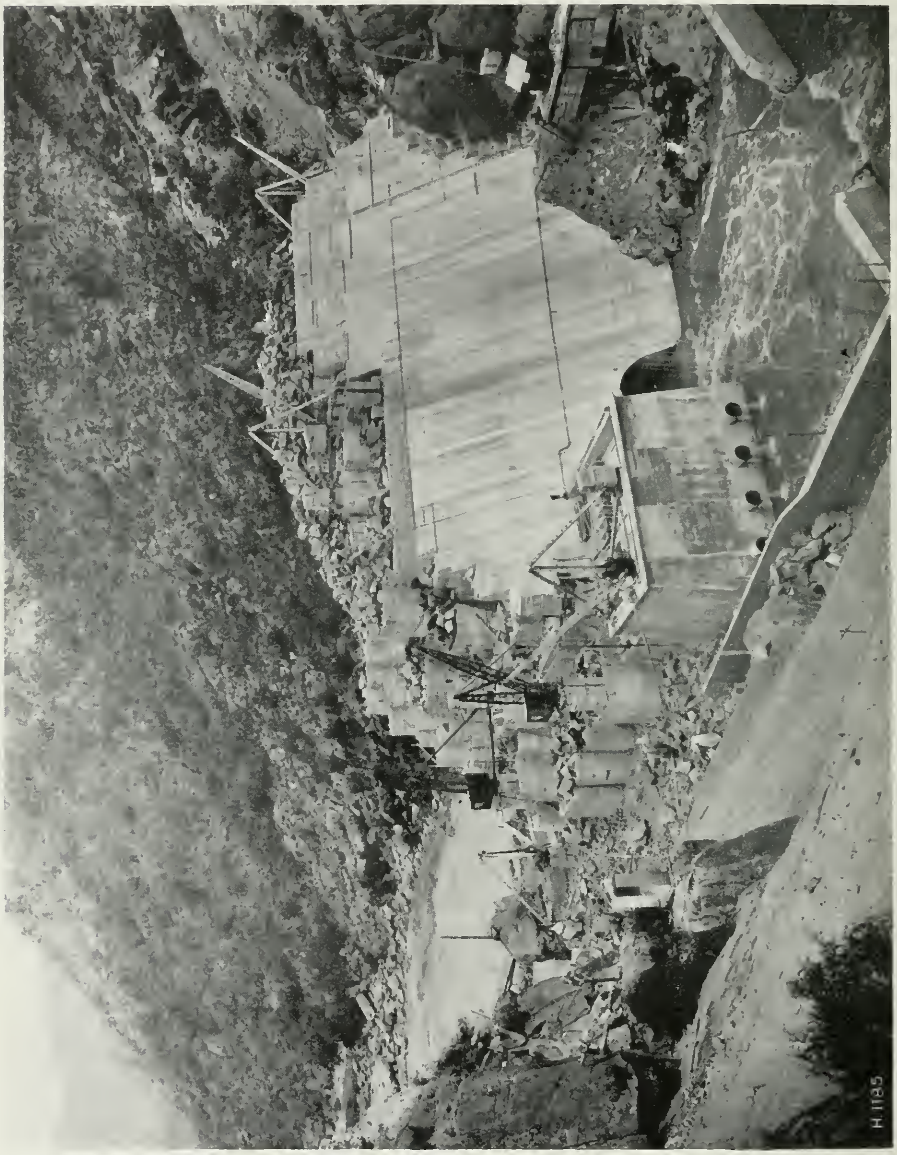
The cyclopean concrete of the retaining wall is concrete work with large stones up to 15 tons in weight embedded in it. It is estimated that there will be in all 50,000 tons of cement required, the whole of which is being locally made and supplied.

In order to handle this vast bulk of masonry, the provision of a special plant for the purpose became necessary. The gorge is spanned along the length of the dam by three cableways, or suspension ropes, of 1,100 ft., or slightly less than a quarter of a mile, in span. These cableways, which are about 400 ft. above the bed of the gorge, handle a maximum load of 15 tons, and are being utilised for conveying concrete, large stones, and for handling and moving the plant on the wall, which consists of an installation of 10-ton cranes. These cableways and the whole of the plant on the work are actuated electrically from a central power station. The cyclopean concrete work, which will form the dam wall, is built up of a series of units, each separate unit representing the average quantity of concrete that can be placed in one full day's work. Each of these units practically represents the space occupied by a good six-roomed cottage. They are designed so as to break-joint in every direction, and in this respect represent a gigantic form of masonry construction. The object aimed at is to have each unit a solid mass without any joints or breaks caused by an intermission of work.

The chiefly responsible engineer, in reporting on the plans for the construction of the dam, was very confident, not only of the capacity of the dam to provide sufficient water for the irrigation area, but also of the capacity of the dam wall to hold up against the tremendous pressure of forty miles of backed-up river waters. He said :

> "As to the security of the structure, the design, tested by Delocre's method, fully complies with the conditions laid down by him. The resultant lines of pressure, for reservoir full and reservoir empty, fall well within the middle third of the mass. The stability and durability of the structure will therefore depend, practically, on the character of the foundations, the quality of the materials employed in the construction, and the character and quality of the workmanship. The computed maximum pressures on the foundations are—at the inner toe, 15 tons, and at the downstream talus, 14·2 tons per square foot; whilst the ultimate resistance to crushing of concrete, made from the available material, and according to the specification adopted, is, from the results of tests made at the Sydney University, 180 tons per sq. ft. The factor of safety in strength is therefore $\frac{180}{15} = 12$; that usually deemed indispensable in such works being from 8 to 10. The resistance to overturning is found by comparing

the resultant of the maximum water pressure against the face of the dam by the weight of its mass, multiplied in each case by the leverage under which it acts. The maximum pressure against a lineal foot of the dam at its highest point is 737·16 tons, acting at 76·66 ft. above the level of its base. Hence the overturning moment is 737·16 tons by 76·66 ft., equal to 56,515·6 foot-tons per lineal foot. The moment of stability of each lineal foot of the dam, at its greatest height, is 1,178·5 tons, the weight of the section, acting at the centre of gravity, which is distant horizontally 103·65 ft. from the down-stream toe. It is therefore 1,178·5 tons by



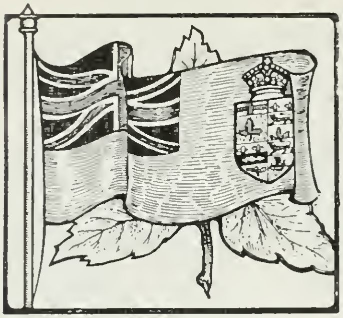
View of the Retaining Wall.
THE BURRINJUCK DAM, NEW SOUTH WALES, AUSTRALIA.

103·63 ft.; equal to 122,126 foot-tons. And the factor of safety in stability is $\frac{122,128}{56,515\cdot 6}$, equal to 2·16. That is, the resistance to movement of the mass is 116 per cent. in excess of the overturning moment."

The Burrinjuck Gorge, looking downstream.
THE BURRINJUCK DAM, NEW SOUTH WALES, AUSTRALIA.

It only remains to say that the Murrumbidgee irrigation scheme is already working, and the first area of over 13,000 acres is being irrigated by settlers.

The Burrinjuck Dam itself is not yet completed to its full height of 240 ft. The engineers from the outset were prepared to allow the inauguration of the scheme to take place as soon as the dam reached a height of 120 ft. As the dam has increased in height, so the irrigation area watered is correspondingly enlarged.



TORONTO AND REINFORCED CONCRETE.

THE NEW BY-LAWS OF 1912 FOR REGULATING REINFORCED CONCRETE CONSTRUCTION

We have from time to time published the Regulations and By-Laws of different countries as regards Reinforced Concrete, and in view of the increasing use of this form of construction in our Colonies, we now present the new By-Laws recently passed for the City of Toronto.—ED.

I.—TERM.

THE term, "reinforced concrete," as used in this by-law, is to be understood to mean an approved concrete mixture reinforced by steel of any shape so combined that the steel will take up the tensional stresses and assist in the resistance to shear, and the construction must be of such a nature that the stresses can be calculated by the accepted formulas of modern engineering practice.

II.—GENERAL.

(1) Before permission to erect any reinforced concrete structure or any structure containing reinforced concrete is issued, complete drawings and specifications, signed by the architect or engineer who designed the structure, and clearly showing all details of the proposed construction, and the size, shape and position of all reinforcing rods, stirrups or other forms of metal, also a certified copy of the computations of the architect or engineer showing how the strength of the different portions of the structure was arrived at, shall be submitted to the department of the city architect and superintendent of building for examination and approval, and, when satisfactory, the approved signed drawings, specifications and static computations shall be filed in the said department until the completion of the work, provided, however, that permission to erect any reinforced concrete structure does not in any manner imply the acceptance of the work or relieve the architect or engineer who designed the structure from full responsibility for the actual construction.

(2) All reinforced concrete construction shall be performed under the personal and constant supervision of either a fully qualified engineer or competent superintendent.

(3) It shall be the duty of the engineer or superintendent in charge of the work to keep an exact record of the progress of each operation, which record shall clearly show the position of, and give the date of, placing of all concrete and the date of removal of forms, a certified copy of which is to be furnished the inspector of buildings weekly; and upon the completion of the work the correctness of the records, which have been furnished weekly, to be certified to and sworn to, if considered desirable, by the person under whose constant supervision the work was carried out.

(4) Inspection by the engineer or superintendent in charge shall cover the following:

(*a*) The materials;

(*b*) The correct construction and erection of the forms and supports;

(*c*) The sizes, shapes, arrangement and fastening in position of all reinforcements;

(*d*) The proportioning, mixing and placing of the concrete;

(*e*) The strength of the concrete by tests of standard test pieces made during the progress of the work;

(*f*) Whether the concrete is sufficiently hardened before the forms and supports are removed;

(*g*) Prevention of injury to any part of the structure by and after removal of the forms;

(*h*) Comparison of dimensions of all parts of the finished structure with the plans.

(5) The execution of reinforced concrete work shall be confided to workmen accustomed to this class of construction, who shall be under the control of a competent foreman, and it is to be distinctly understood that under no conditions will the engineer or superintendent in charge of the work be permitted to act in the dual capacity of both inspector and foreman.

III.—FORMS.

(1) All forms shall be built rigid, plumb and true, thoroughly braced and with tight joints, so that no appreciable part of the concrete mixture can escape, and their interior dimensions must conform to the dimensions of the concrete sections shown upon the approved plans. All forms shall be supported so as to carry the dead load of the construction as a liquid without spring or deflection, and any form that becomes twisted or warped shall be correctly adjusted before concrete is placed therein. Building paper will not be allowed in forms for any purpose whatsoever.

(2) If forms are to be hung from steel beams the hangers and the method of hanging shall be subject to the approval of the inspector of buildings, and no hangers that are difficult to remove shall be used.

(3) The forms for beams, girders and lintels shall be so designed that at least one side of each beam or girder may be removed without disturbing the bottom portion of the form or its supports.

(4) All posts supporting forms for slabs, beams or girders must rest upon wedges which can be loosened and removed without producing undue strains in the floor or roof system.

(5) Posts which are to be supported from the ground must be provided with footings of sufficient area to do away with any liability of appreciable settlement owing to insufficient foundation.

(6) All column forms shall have an opening left at the bottom to enable cleaning out and adjustment of the steel to be attended to, which opening is not to be closed until the column is to be poured.

(7) All shavings, chips, sawdust, ice or other foreign matter must be removed from within all forms before concrete is placed therein.

IV.—CEMENT.

(1) Only Portland cement will be accepted in reinforced concrete construction, and this cement must comply in all respects with the requirements of the standard specifications of the city engineer's office of the City of Toronto, which are as follows:

(*a*) The specific gravity of the cement shall not be less than 3·10. Should the test of cement as received fall below this requirement, a second test may be made upon a sample ignited at a low red heat. The loss in weight of the ignited cement shall not exceed 4 per cent.

(*b*) It shall leave by weight a residue of not more than 8 per cent. on the No. 100 and not more than 25 per cent. on the No. 200 sieve.

(*c*) It shall not develop initial set in less than thirty minutes, and must develop hard set in not less than one hour, nor more than ten hours.

(*d*) The minimum requirements for tensile strength for briquettes 1 sq. in. in cross-section shall be as follows, and the cement shall show no retrogression in strength within the periods specified:

Age.	Strength.
Neat Cement.	
24 hours in moist air	175 pounds
7 days (1 day in moist air, 6 days in water)	500 ,,
28 days (1 day in moist air, 27 days in water)	600 ,,
One part cement, three parts standard Ottawa sand.	
7 days (1 day in moist air, 6 days in water)	200 ,,
28 days (1 day in moist air, 27 days in water)	275 ,,

(*e*) Pats of neat cement about 3 in. in diameter, ½ in. thick at the centre, and tapering to a thin edge, shall be kept in moist air for a period of twenty-four hours.

(*f*) A pat shall then be kept in air at normal temperature and observed at intervals for at least twenty-eight days.

(*g*) Another pat shall be kept in water maintained at as near 70° F. as practicable and observed at intervals for at least twenty-eight days.

(*h*) A third pat shall be exposed in any convenient way in an atmosphere of steam above boiling water in a loosely closed vessel for five hours.

These pats, to satisfactorily pass the requirements, shall remain firm and hard and show no signs of distortion, discoloration, checking, cracking or disintegrating.

(*i*) The cement shall not contain more than 1·75 per cent. of anhydrous acid (SO_3), nor more than 4 per cent. of magnesia (MgO).

(2) No cement shall be used in any reinforced concrete work unless it has been tested either at the manufactory or at some acceptable laboratory, and a certified report of the test filed with the inspector of buildings.

(3) All such tests shall be made in not more than car-load lots, or in smaller quantities if so directed by the inspector of buildings, and when laboratory tests are to be made the cement shall be delivered at the job at least two weeks before it is required for use, so as to allow ample time for testing.

(4) Proper care is to be taken to separate lots delivered, so that they can be easily identified if found unsatisfactory, and a suitable place must be provided for the storage of all cement. No cement shall be used that has absorbed sufficient moisture to cause it to granulate or become lumpy when thoroughly dried.

(5) Any cement without the makers' name and brand on the barrel or package will be rejected without test, as will also any cement in barrels or packages bearing other than the makers' name and brand.

(6) The inspector of buildings may require a certificate from the manufacturer to the effect that the cement has been seasoned or subjected to aeration for at least thirty days before leaving the works.

(7) All tests shall be made at the expense of the contractor, and all cement or other rejected material must be immediately removed from the job and the vicinity.

V.—Aggregates.

(1) Extreme care shall be exercised in selecting the aggregate for mortar and concrete, and careful tests must be made where any doubt exists of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density or a minimum percentage of voids.

(2) Fine aggregates shall be uniformly graded from coarse to fine and consist of sand, crushed stone or gravel screenings, passing when dry a screen having ¼-in. diameter holes, and not more than 6 per cent. passing a sieve having 100 meshes per lineal inch. It shall be of clean silicious material free from vegetable loam and other deleterious matter.

(3) Mortars composed of one part Portland cement and three parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength of at least 70 per cent. of the strength of 1 : 3 mortar of the same consistency made with the same cement and standard Ottawa sand.

(4) Coarse aggregate shall consist of inert material, such as crushed stone or screened gravel, which is retained on a screen having ¼-in. diameter holes. The particles shall be clean, hard, durable and free from dust or other deleterious material, and the maximum size shall pass in any direction through a 1-in. diameter ring. A graduation of the size of the particles from fine to coarse will be considered advantageous.

(5) Samples of all aggregates must be submitted to the inspector of buildings, and his approval of them obtained before being used in any work.

VI.—Reinforcement.

(1) All steel used for reinforcement shall be medium steel made by the open hearth or Bessemer process, and be rolled from new billets. No re-rolled steel will be allowed. The steel shall have an ultimate tensile strength of not less than 60,000 lb. per sq. inch of net cross-section, and an elastic limit of at least one-half of this ultimate

strength, and for sizes up to $\frac{3}{4}$ in. in diameter shall bend cold 180° around a diameter equal to the thickness of the piece tested without sign of fracture on the outside of bent portion.

(2) For sizes greater than $\frac{3}{4}$ in. in diameter it shall bend cold 180° around twice its own diameter without sign of fracture on the outside, except in cases where all bends necessary in the construction are to be made hot, in which case the steel shall bend cold 90° around its own diameter without sign of fracture.

(3) All material tested to destruction shall show a uniform silky fracture, and all reinforcing steel shall be free from checks, cracks, flaws or other imperfections, also free from paint, oil, dirt, grease or heavy rust or scale.

(4) If deformed bars are used, only the net section—the section exclusive of all projections—of the area of such bars shall be considered effective.

(5) Tests of the steel proposed to be or being used in any reinforced concrete structure shall be made at the expense of the contractor upon the request of the inspector of buildings, and such tests shall be made at a place satisfactory to him and under the supervision and direction of the architect, engineer or superintendent in charge of the work.

(6) A certified copy of the report of all such tests to be filed with the inspector of buildings immediately after being made.

VII.—Mixing.

(1) The ingredients of concrete shall be thoroughly mixed dry, after which the proper amount of water shall be added and the mixing continued until the cement is properly distributed and the mass uniform in colour and homogeneous.

(2) Methods of measurement of the proportions of the various ingredients, including water, in each batch shall be used which will secure separate uniform measurements at all times. The use of boxes specially constructed for the purpose will be preferred, but the use of iron wheelbarrows may, with the consent of the Inspector of Buildings, be used for measuring stone and sand.

(3) All concrete materials shall be measured loose.

(4) All concrete must be machine mixed, using a batch mixer of an approved design if the amount of concrete in the work exceeds fifty (50) cubic yards. Continuous mixing machines will not be allowed.

(5) A competent foreman must be in constant attendance at the mixer to give his approval of every batch which leaves the machine.

(6) When the amount of concrete does not exceed fifty (50) cubic yards, the mixing may be done by hand on a smooth watertight platform not less than 14 ft. by 12 ft. square, and having a raised rim around the edge at least 3 in. in height. The fine aggregate shall be first evenly spread on this platform and the proper proportion of cement shall then be spread over the aggregate, when the ingredients shall be thoroughly mixed with hoes or shovels, then spread out evenly over the platform while in a dry state. Clean water shall then be applied and the cement and fine aggregate thoroughly mixed until a mortar of the proper consistency is formed, which shall be evenly spread over the platform. The stone or gravel after being wetted shall then be evenly spread over the mortar and the whole mass thoroughly mixed with shovels and hoes by being turned over and worked at least three times, not counting the shovelling off the platform. No more than $\frac{3}{4}$ of a cubic yard shall be mixed in any one batch.

(7) The materials must be mixed wet enough to produce a concrete of such consistency as will flow into the forms and about the metal reinforcement and which at the same time can be conveyed from the mixer or platforms to the forms without separation of the coarse aggregate from the mortar.

(8) Retempering mortar or concrete, *i.e.*, remixing with water after it has partially set, will not be permitted.

VIII.—Placing Concrete.

(1) Concrete shall be deposited in the forms as rapidly as possible after leaving the mixer, and under no circumstances shall concrete be used that has attained initial set before final placing, nor shall such concrete be returned to the mixer.

(2) When placing concrete is once started it shall, if possible, be carried on as a continuous operation until the pouring of the section or panel is completed.

(3) When being deposited or poured the concrete shall be agitated continuously with suitable tools such as a straight shovel or slicing tool kept moving up and down until all the ingredients have settled to their proper place by gravity, and the surplus water has been forced to the surface. Filling the forms completely and puddling afterwards will not be permitted. In placing the concrete the work shall be so laid out that the partly set concrete shall not be subjected to shocks from men hauling or wheeling material over it.

(4) Before placing concrete care shall be taken to see that the forms are constructed as hereinbefore specified for and thoroughly wetted, and the space to be occupied by the concrete free from débris.

(5) When the placing of concrete is suspended all necessary grooves for joining future work shall be made before the concrete has had time to set. When work is resumed concrete previously placed shall be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts of fine aggregate.

(6) Care must be taken to stop work at such a point that the joint formed when the work is resumed will have the least possible effect on the strength of the structure. Footings shall be cast to their full depth at one operation.

(7) Construction joints in beams and girders shall be vertical and at a point midway between supports, unless a beam is located at this point, in which case the joint shall be offset at a distance equal to twice the width of the beam.

(8) Construction joints in slabs shall be near the centre of the span. No joint will be allowed between slab and beam or girder.

(9) Any concrete which may run past the bulkheads must be cleaned up and removed before the concreting of the next section is started.

(10) Where brackets are used the brackets will be considered a part of the beam or girder.

(11) All columns are to be poured a sufficient length of time ahead of the floor construction to allow the concrete in the column to properly set up. The pouring of a column must be in one continuous operation to the bottom of the beam or girder it supports, and during the pouring the concrete shall be well stirred or puddled with a long rod or tamp to expel all bubbles of air and prevent voids and honeycombing. Joints in columns shall be perpendicular to the axis of the column.

(12) In slab construction the finish must be laid integrally with the rest of the slab, or it will not be considered as part of the slab in calculating the strength of same.

(13) Whenever possible the edges of girders, beams, and columns shall be chamfered and the sides of beams and girders splayed in order that the forms may be more easily removed.

(14) All reinforcing steel shall be accurately located in the forms and secured against displacement, and the lateral spacing of the steel in lintels, beams, or girders shall not be less than two and one-half diameters centre to centre.

(15) No concrete shall be poured until the engineer or superintendent in charge has examined the particular section in which it is desired to proceed with this portion of the work and has expressed himself satisfied with the structural condition and cleanliness of the forms and the placing, splicing and fastening in its proper position of the steel reinforcement.

(16) When concreting is carried on in freezing weather the material must be heated and such provisions made that the concrete can be put in place without freezing. The use of frozen, lumpy sand or stone depending on hot water used in mixing to thaw it out will not be permitted.

(17) All concrete shall be kept at a temperature above freezing until it has thoroughly hardened, and all concrete which is frozen shall be removed.

(18) Concrete placed in warm weather, or which from any other cause is exposed to premature drying, shall be kept thoroughly wet during the first week after being put in place.

(19) Concrete shall not be placed in water unless unavoidable, but when this cannot be avoided, unusual care must be exercised to prevent the cement from being floated away, also the formation of laitance. The concrete shall also be deposited through a metallic tube or from a bucket having a bottom dump, and care shall be taken to keep the surface of the concrete as nearly horizontal as possible. The pro-

portion of cement in concrete deposited under water shall be 25 per cent. in excess of that required for similar work deposited in the ordinary way.

IX.—REMOVAL OF FORMS.

(1) Forms shall not be removed until the concrete is thoroughly set and is of sufficient strength to carry its own weight together with whatever live load is liable to come on the construction.

(2) No form shall be removed without the approval of the architect or engineer or the superintendent in charge of the structure.

(3) The original supports for beams and girders must remain in place for at least twenty-one days after the concrete has been poured, and before these supports are interfered with the entire forms must be removed from supporting columns and the sides of beams and girders stripped so as to enable a thorough examination of the concrete to be made.

(4) The form shall not be removed from columns in less than six days, and forms supporting floor slabs shall not be removed in less than ten days after the concrete has been poured. The sides of forms for beams and girders may be removed at the same time as the forms supporting the floor slabs.

X.—GENERAL ASSUMPTIONS.

(1) As a basis for calculations for the strength of reinforced concrete construction, the following assumptions shall be made:—

(*a*) A plane section before bending remains plane after bending.

(*b*) The modulus of elasticity of concrete in compression within the usual limit of working stresses is constant.

(*c*) In making calculations the tensile stress of concrete shall not be considered.

(*d*) The steel shall take all the tensile stresses.

(*e*) Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials are, therefore, stressed in proportion to their moduli of elasticity and their distance from the neutral axis.

(*f*) The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete shall be assumed to be fifteen for concrete with a crushing strength of 2,000 lb. per sq. in. after hardening for twenty-eight days.

(*g*) Initial stress in the reinforcement due to contraction or expansion in the concrete may be neglected.

(2) The span length of beams and slabs shall be taken as the distance from centre to centre of supports, but shall not be taken to exceed the clear span plus the depth of the beam or slab. Brackets shall not be considered as reducing the span.

(3) The dead load shall include the weight of the structure and all fixed loads and forces.

(4) The live load shall include all loads and forces which are variable.

(5) The weight of reinforced concrete shall be taken as 150 lb. per cu. ft.

XI.—BENDING MOMENTS.

(1) The bending moment for slabs and beams, when not continuous over supports, shall be taken at not less than $\frac{WL}{8}$

(2) When slabs and beams are built continuous over two or more supports the bending moment shall be taken at not less than $\frac{WL}{10}$

(3) When slabs and beams are continuous over one support only the bending moment shall be taken at not less than $\frac{WL}{9}$

(4) In the case of square panels reinforced in both directions and continuous over all supports the bending moment of the slab shall be taken at not less than $\frac{WL}{20}$ provided, however, that when square floor slabs reinforced in both directions are contiguous to walls, the bending moment of the slab shall be taken at not less than $\frac{WL}{18}$

W = Total uniformly distributed load.
L = Span of beam or slab.

(5) The length of a slab in which reinforcement in both directions will be allowed for is limited to one and a half times the width. The proportion of load on a slab up to this limit which will be considered as being transferred to the side and end supports will be that obtained by using the following formulas:

(a) Load on longer supports $= \dfrac{L^4}{L^4 + B^4}$

(b) Load on shorter supports $= \dfrac{B^4}{L^4 + B^4}$

L = Length of span of slab.
B = Breadth of span of slab.

(6) The amount of reinforcing steel may be gradually reduced to 75 per cent. of that calculated per foot in width, commencing at the quarter point and continuing to the support. The reinforcement spanning the shortest direction shall be placed below the reinforcement spanning the longer direction.

(7) In all cases where reduction in the bending moment is allowed on account of continuous action, sufficient steel reinforcement must be provided in the top of the slab or beam over the support to meet the requirements of the accepted theory of flexure, and this upper reinforcement shall extend on both sides a sufficient distance beyond the centre of the support to develop adhesion equal to at least the strength of the upper bars on the basis of the allowable unit stresses.

(8) If the girders, beams, and slabs are poured in one continuous operation, then the girders or beams may be treated as T-beams with a portion of the slab acting as a flange. In no case shall the overhanging width on each side of the girder or beam exceed four times the thickness of the slab and the total width of the flange shall not exceed five times the width of the girder or beam.

(9) In beam and slab construction an effective metallic bond shall be provided at the junction of the beam and slab, and where reinforced concrete girders carry reinforced beams the portion of the floor slab acting as flange to the girder must be sufficiently reinforced with bars near the top, at right angles to the girder, to enable local loads to be transmitted directly instead of through the beams to the girder, thus avoiding an integration of compressive stresses due to simultaneous action as floor slab and girder flange.

(10) In the design of T-beams acting as continuous beams, due consideration shall be given to the compressive stresses at the support at the bottom of the beam.

(11) When the overall vertical distance of the tension members of a girder or beam is greater than one-sixth of the total depth of the girder or beam, the stresses in members shall be computed in proportion to the distance from the neutral axis.

(12) Shrinkage and thermal stresses must be provided for by the introduction of steel.

XII.—Proportion of Concrete and Allowable Stresses on Concrete and Steel.

(1) All concrete for columns, girders, beams, slabs, walls, fireproofing or piles shall be composed of materials meeting the requirements of these regulations, and be mixed, as hereinbefore specified, in the proportion of not less than one part of cement to two parts of fine aggregate and four parts of coarse aggregate, or in such other proportions as may be necessary to make the resistance of the mixture to crushing not less than two thousand (2,000) lb. per sq. in. after hardening for twenty-eight days.

(2) Tests to determine this value must be made at the expense of the contractor when required by the Inspector of Buildings, at a place and by a person satisfactory to him and under the direct supervision of either the said Inspector of Buildings or a person representing him.

(3) When the proportion of cement is increased and the quality of aggregates hereinbefore called for used, an increase may be made on the allowable working stresses proportional to the increase in compressive strength at twenty-eight days as determined by actual tests, but this increase shall not exceed 25 per cent. On this basis the following maximum stresses for 1 : 2 : 4 concrete will be allowed in construction:—

(a) Compression in extreme fibre of girders, beams and slabs, 600 lb. per sq. in.
(b) Direct compression, 450 lb. per sq. in.

(*c*) Shearing stress, when diagonal tension is not resisted by steel, 40 lb. per sq. in.

(*d*) Shearing stress, when all diagonal tension is resisted by steel, not to exceed 100 lb. per sq. in. of effective cross-section.

Members of web reinforcement shall be designed so as to adequately take up all involved stresses throughout their entire length. They shall not be spaced to exceed three-fourths of the depth of the beam in that portion where the web stresses exceed the allowable shearing value of the concrete.

Web reinforcement, unless rigidly attached, shall be placed at right angles to the axis of the beam and looped around the extreme tension member.

(*e*) For T-beams the width of the stem only shall be used in calculating the shear.

(*f*) The resisting moment in inch-pounds of a rectangular beam shall be taken as $95\ B\ D^2$. (Moment arm $= \cdot 88$ D.)

B = Breadth of beam in inches.
D = Effective depth of beam in inches.

(*g*) Adhesion to plain steel bars, 60 lb. per sq. in. of surface of bar.

(*h*) Adhesion to deformed steel bars, up to 100 lb. per sq. in. of surface of bar may be permitted.

(*i*) For compression in columns with not less than 1 per cent. and not more than 4 per cent. of longitudinal reinforcement the safe load shall be computed as follows:—

Safe load (in lb.) $= 450\ (Ac + 15\ As)$.
Ac = Net cross-sectional effective bearing area of concrete in square inches.
As = Cross-sectional area of longitudinal reinforcement in square inches.

(*j*) For columns reinforced with both longitudinal steel and spirally wound hooping when the volume of hooping is equal to at least 1 per cent. of the volume of the enclosed concrete and the longitudinal reinforcement is not less than 1 per cent., and not more than 4 per cent. of the cross-sectional area, the safe load shall be computed as follows:—

Safe load (in lb.) $= 650\ (Ac + 15\ As)$.
Ac = Net cross-sectional area of concrete enclosed in hooping in square inches.
As = Cross-sectional area of longitudinal reinforcement in square inches.

(4) The foregoing allowed stresses shall be used only where the unsupported length of the column is not greater than fifteen times the least diameter of effective bearing area. Where the length exceeds this limit the allowable stress shall be reduced according to approved standard formula.

(5) Bending stresses due to eccentric loads on columns shall be provided for by increasing the section of concrete or steel until the maximum stress on a column will not exceed the allowable working stress.

(6) The least diameter of effective bearing area is to be understood to mean the distance from inside to inside of hooping in columns with spiral hooping, and in others from outside to outside of longitudinal reinforcement.

(7) The pitch of spiral reinforcement shall not be greater than one-sixth of the effective diameter of a column, and in no case more than 3 in., and adequate means must be provided to hold it in place so as to form a column, the core of which will be straight and well centred.

(8) When columns are reinforced with longitudinal rods only, the reinforcement shall be tied together with bands not less than $\frac{1}{4}$ in. in diameter, placed not more than the effective diameter of the column apart, or in any case not more than 12 in. The bands to be jointed in such a manner as to do away with any liability of the reinforcement spreading, and shall be wired to each longitudinal rod.

(9) The percentage of reinforcement in any column, regardless of the load carried by it, is not to be less than 1 per cent. of the effective cross-sectional area. Reinforcement to be in all cases placed at corners of square or rectangular columns, the minimum size of which shall be $\frac{9}{16}$ in. round or $\frac{1}{2}$-in. square steel rods.

(10) No column shall have less than 64 sq. in. of effective area or a dimension of less than 8 in.

(11) When longitudinal reinforcing rods in columns have an area in excess of $1\frac{1}{4}$ sq. in. the ends shall be faced or milled off normal to the longitudinal axis, and such

rods shall have full and perfect bearings and a tight fitting sleeve or other approved appliance shall be provided at each joint to keep the rods in their proper position.

(12) When longitudinal reinforcing rods in columns have an area of less than $1\frac{1}{4}$ sq. in. the rods may be lapped and securely wired together, the splice to be of a sufficient length to develop by adhesion the strength of the rod.

(13) All butted joints shall be made at the floor levels or at points where the columns may be considered as fixed, and the centre of lapped joints shall not be more than 1 ft. above floor levels or points at which rigid lateral support is afforded the columns. The ends of all rods at the base of columns shall be made smooth and right angled from the longitudinal axis, and such rods shall have a perfect bearing on a steel plate or casting of sufficient size and strength to distribute the load which the column supports to such an extent that the allowable compressive stress per square inch on the material under this plate shall not be exceeded, or, in lieu of the plate, the stress may be distributed in concrete footings to the required extent by means of dowels of sufficient length and area to sustain the weight by adhesion of the concrete to the steel, without exceeding the specified limit for such in this regulation. The tops of all dowels to extend to the top of the footing and be made perfectly level and smooth to receive the longitudinal reinforcing rods, and the joint between the dowels and the rods to be made with a pipe sleeve which is to be grouted into the footing at least 6 in. and extend 6 in. above the top of same, and the reinforcing rods after being secured in their proper position are to be solidly grouted into the sleeves with liquid Portland cement.

(14) The area required in footings which have to support columns having both spiral hooping and vertical reinforcement shall be obtained in a similar manner to that immediately above specified for columns with vertical reinforcement only. The longitudinal reinforcing bars in all columns shall be straight, and sufficient metallic lateral support shall be provided to keep them in their proper place until the concrete in the column has set.

(15) The allowable stress on steel in tension shall be sixteen thousand (16,000) lb. per square inch.

(16) Where it is necessary to introduce steel to resist compression in girders, beams, or slabs, the compressive stress per square inch allowed on such steel shall not exceed fifteen (15) times the computed compressive stress in the concrete at the same distance from the neutral axis. All such steel shall be anchored into the mass of concrete in such a way as to prevent any possibility of buckling.

XIII.—PROTECTION.

The minimum protection for steel reinforcement, which is to be taken, is the distance from the surface of the steel to the nearest concrete surface, and shall be:

(*a*) For girders and columns, 2 in.
(*b*) For beams and lintels, $1\frac{1}{2}$ in.
(*c*) For floor and roof slabs, 1 in.

XIV.—PLAIN AND REINFORCED CONCRETE WALLS.

(1) Concrete walls without reinforcement, other than those of foundation or basement, shall be constructed with concrete of the same composition and quality in all respects to that hereinbefore specified for reinforced concrete, and shall be the full thickness prescribed in this By-law for brick walls.

(2) Buildings having a complete skeleton construction of steel or of reinforced concrete construction, or a combination of both, may have exterior walls of reinforced concrete at least 8 in. thick; provided, however, that such walls shall support only their own weight, and that such walls shall have both vertical and horizontal steel reinforcement on both sides, the rods to be placed not more than 18 in. apart from centres, and be securely wired together at each intersection and rigidly connected with columns and girders. All rods shall be lapped a sufficient length to develop their full stress by the allowable unit stress for adhesion. Additional rods shall be set around openings, the vertical ones to be wired to the nearest horizontal ones, and the horizontal rods at top and bottom of openings wired to the nearest vertical ones. In all cases the percentage of steel reinforcement to be provided shall be sufficient to enable the wall to safely resist a uniformly distributed horizontal pressure of 30 lb. per square foot

acting on either side of it, but in no instance shall the percentage of reinforcing metal be less than one-quarter of 1 per cent. in each direction. The thickness of a reinforced concrete curtain wall shall not be less than one-eighteenth of the unsupported height.

XV.—Tile and Reinforced Concrete Joist Construction.

(1) When tile and reinforced concrete joist construction is used for floors the tile shall be sound, hard burned, free from shrinkage cracks, of uniform size and corrugated in a horizontal direction upon the sides.

(2) When concrete tile and reinforced concrete joist construction is used for floors the tile shall be sound, hard, well seasoned, corrugated in a horizontal direction upon the sides, and made to comply in all respects with the requirements of the Inspector of Buildings.

(3) The reinforced concrete joists shall be parallel and in perfect line and of sufficient width to develop the steel, which steel is to be protected from fire with concrete of a similar mixture and thickness to that hereinbefore specified for concrete beams. Should concrete be required on the top of the tile to take compressive stress, it must be not less than 2 in. in thickness, and must be poured at the same time as the joists, and be of the same mixture. The tile shall be thoroughly soaked with water before the pouring of the concrete is commenced. In this class of construction the dead and entire live loads will be assumed to be carried by the reinforced concrete joists.

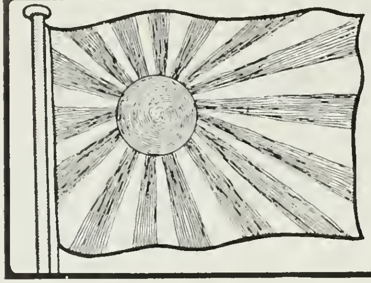
XVI. to XIX.—Administrative.

Upon the completion of a reinforced concrete building the architect or engineer who designed the structure shall issue a signed certificate and have the same posted in a conspicuous place on each floor, stating the safe capacity per square foot of floor space, if the entire floor is designed to carry the same load. If, however, the floor is designed to carry different loads, each section must be provided with a certificate stating the capacity of that particular section and describing the limits thereof.

Load tests shall be made by, and at the expense of, the contractor when required by the Inspector of Buildings, under his direction and in his presence, or in the presence of his representative, on any portion of a reinforced concrete structure within a reasonable time after erection. Such tests shall show that the construction will sustain a load of twice the live load for which it was designed, without causing any permanent deformation.

No system of reinforced concrete shall be used which is not capable of design and investigation in accordance with the principles laid down in these regulations.

Section 111 of By-law No. 4,861, being "A By-law for regulating the erection and to provide for the safety of buildings," is hereby repealed.



A NEW FORM OF CONCRETE RIVER-BANK PROTECTION.

We give below some short particulars of a new reinforced concrete mattress for river-bank protection, which has been employed in Japan with considerable success.—ED.

The method of river-bank protection here described is intended to take the place of the ordinary fascine type which has been widely used in Japan. The mattress was first used on the Yubari River in 1909, where it has been under careful observation. The main body of the mattress consists of reinforced concrete blocks knitted together by passing metallic wire of the proper diameter through two holes previously made in the centre plane of each block, thus forming a flexible mattress, each block being so set as to break joints. The size of the mattress may be varied according to necessity by the use of a scaffold.

The advantages claimed for the mattress are its comparative cheapness, its strength and durability, its adaptability to the deformation of the river bed on account of its flexibility, its stability even on a steep slope, the small resistance it offers to the stream on account of its smoothness and thinness, and lastly the simplicity of its construction and setting.

The blocks used in work so far undertaken had a thickness of 6 in., and were 2 ft. long by 6 in. wide. They are made of 1 : 3 : 6 Portland cement, sand, and gravel concrete, and in the experimental work both a wet and a dry mix were employed.

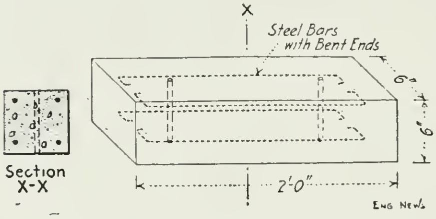


Fig. 1. Details of Block Used in Reinforced Concrete Mattress.

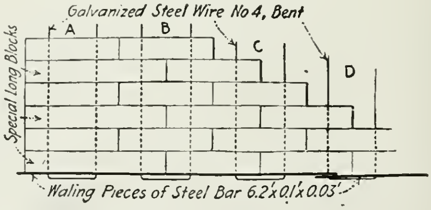

Galvanized Steel Wire No 4, Bent
A
B
C
D
Special long Blocks
Waling Pieces of Steel Bar 6.2'x0.1'x0.03'

Fig. 2. Method of Joining Blocks to Form Mattress.

The reinforcement of each block in the experimental work consisted of No. 12 galvanized steel wire, somewhat shorter than the length of the blocks, with the ends bent as shown in *Fig.* 1. Four reinforcing bars are inserted lengthwise for each block, the object of the reinforcement being not only to strengthen the block, but to prevent splitting off in separate pieces should a crack occur. The method of joining the blocks is shown in *Fig.* 2. The string or wire for knitting together the separate blocks is a No 4 galvanized steel wire for fresh water, and it is better to employ one long continuous wire bent double so as to avoid joints in the wire as much as possible. The blocks are strung successively on to these wires, and after the last block has been placed, the upper ends of each wire are tied together. There is a waling-piece of flat steel bar 6 to 10 ft. long, 1⅛ in. wide and ⅜ in. thick. For salt water this bar may be of copper or aluminium. This waling-piece serves as a form for spacing the blocks and to protect them against the shock of the drift, and so distributing the pressure over a wide area. The forms for the blocks were made of metal in the experimental work and so arranged that forty blocks could be cast in each form. Our illustration, *Fig.* 3, shows a single steel mould. The block is pushed up from the bottom of the mould as soon as the ramming of the concrete is finished. *Fig.* 4 shows a steel combination mould for casting forty reinforced concrete blocks at once.

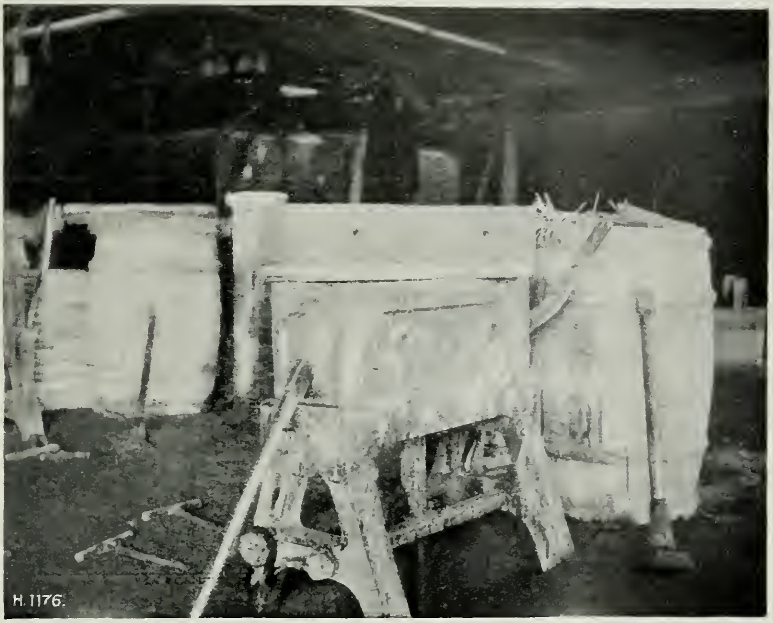


Fig. 3. Single Steel Mould for Block.

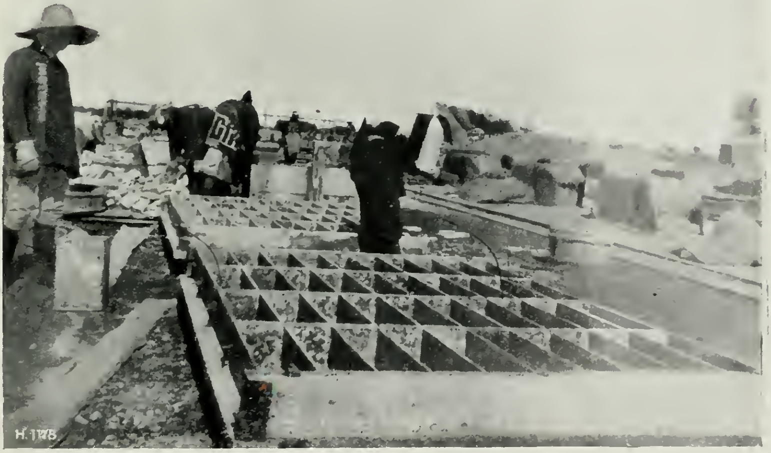


Fig. 4. Steel Combination Mould for Casting 40 Blocks.
Reinforced Concrete Mattress.

Two methods of scaffolding have been employed by the designer in his experimental work. The first form used in Japan was what is known as the "free log scaffold," of which an illustration is given in *Fig.* 5. This scaffold consists of slender logs of suitable length spaced 1 ft. apart and tied at their upper extremities to a horizontal waling-log, which is supported by stakes temporarily driven into the bank. The lengths of the logs composing the main part of the scaffold are so fixed that, when sunk, their lower extremities will reach the river bottom where the lower end of the mattress is intended to be located when set. In this scaffold the lower extremities of the logs are free. This form of scaffold can be used in water up to 15 ft. in depth.

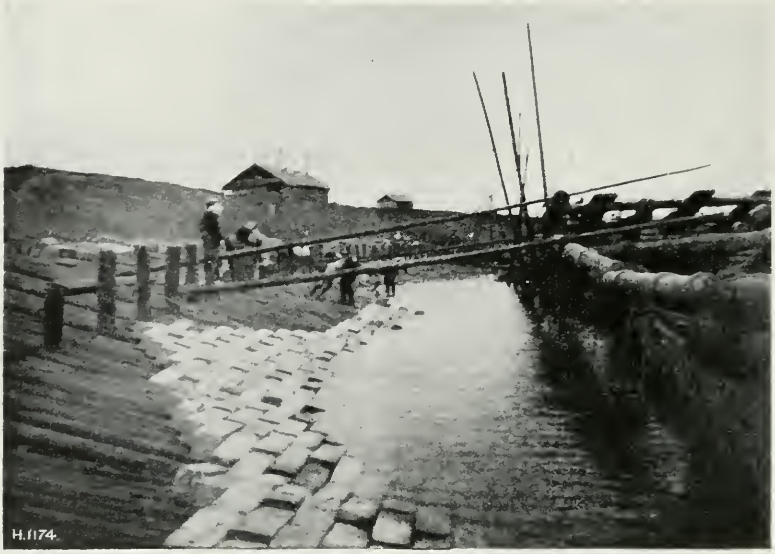


Fig. 5. Showing Free Log Scaffold.

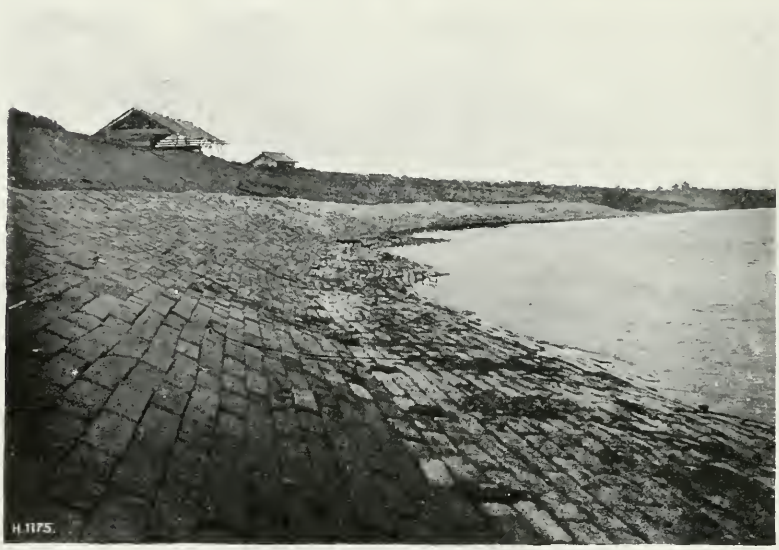


Fig. 6. Showing River Bank directly after Mattress has been laid.
REINFORCED CONCRETE MATTRESS.

The second form of scaffold is known as the "pontoon hung log scaffold," and is in the main a scaffold of the "free log" type hung at its lower extremity to several pairs of pontoons. It is specially suitable for work where the water is of a greater depth than 15 ft.

In treating, the river bank, the natural bank, on and in front of which the mattress is to be set, is at first graded to a proper slope above low-water line and the excavated material thrown into the water directly in front of the excavated bank. In this way the portion of the bank below low-water line is irregularly covered with the loose material excavated from the upper part; the mattress rests on this irregularly formed surface, subject, of course, to further scouring and settling down.

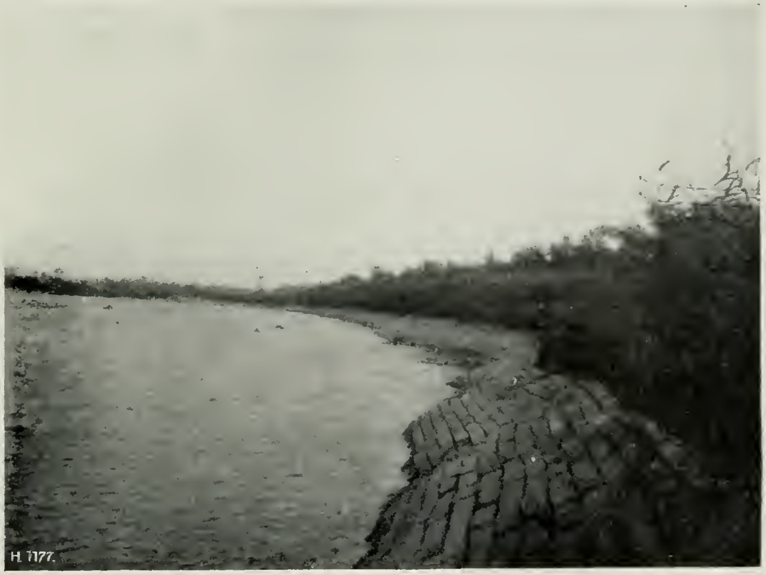
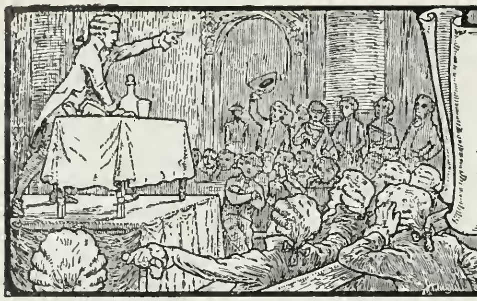


Fig. 7. Showing River Bank 12 months after Mattress has been laid.
REINFORCED CONCRETE MATTRESS.

As mentioned above, this form of protection was first tried experimentally on a section of the Yubari River, Japan, and subsequent investigations and tests have shown that it is effective and satisfactory.

Fig. 6 shows a section of the bank just after it has been covered by the mattress, and *Fig.* 7 shows the same bank a year later.

This mattress, as also the moulds for the concrete blocks, are the invention of Mr. B. Okazaki, Chief Engineer of the Ishikari River Improvement Works, Sapporo, Japan, to whom we are indebted for most of our illustrations, and the remaining particulars here given were taken from the *Engineering News*, U.S.A.



RECENT VIEWS ON CONCRETE AND REINFORCED CONCRETE.

RECENT PAPERS & DISCUSSIONS.

It is our intention to publish the Papers and Discussions presented before Technical Societies on matters relating to Concrete and Reinforced Concrete in a concise form, and in such a manner as to be easily available for reference purposes.

The method we are adopting, of dividing the subjects into sections, is, we believe, a new departure.—ED.

THE INSTITUTION OF CIVIL ENGINEERS.

The following are abstracts of two Papers read at the Ordinary Meeting of the Institution of Civil Engineers on November 12th, 1912:—

THE CONSTRUCTION OF THE NEW DOCK AT METHIL.

By B. HALL BLYTH, Junr., Assoc.M.Inst.C.E.

After some introductory remarks regarding the port of Methil and the export trade in coal from Fife, the author continued to give a detailed account of the various improvements made to cope with the ever-increasing shipping trade. Until 1887 Methil had no docks, but in that year the first dock was opened, and a considerable quantity of coal was shipped. Every subsequent year showed a very large increase in the shipments, and before many years the North British Railway Company, who had purchased the first dock from Mr. Erskine Wemyss, of Wemyss Castle, built a second and larger dock, which was finished in 1899. For a few years these two docks were able to cope with the coal traffic, but lately, with the great increase in the output, the accommodation proved inadequate, and it was determined to build still another dock, which would be sufficient in every way for the trade for many years to come. Plans for this new dock were prepared by Messrs. Blyth and Westland, of Edinburgh, and the contract for its construction was let at the end of 1907 to Messrs. Robert McAlpine and Sons, of Glasgow.

The new dock is situated to the east of the two old docks, and is 16½ acres in extent, with a depth of water of 32 ft. at high water of ordinary spring tides, and gates 80 ft. wide. It is approached by a channel 120 ft. wide and 1,800 ft. long, running along the outside of the sea-wall which enclosed the two old docks. This channel and the dock are situated almost entirely on the foreshore, and are enclosed by a sea-wall, running along and outside low-water mark, from the land end of the existing pier or breakwater to a point near the mouth of the River Leven. This wall is nearly a mile long; it is built for three-fourths of its length of concrete, and continues with a pitched slope where it is well above low water.

The entrance to the present docks, which is well protected, is used for the new dock, a part of the existing pier having been removed across the line of the new channel.

The whole area reclaimed by the sea-wall amounts to 44 acres, the excavation for the dock and channel being used to level up the rest of this area to quay-level for siding accommodation for the dock.

For loading the coal the dock is equipped with hydraulic hoists, of which there are six at present; but provision is made for three more when they are required. With all these hoists and those in the old docks, no difficulty will be found in shipping 10,000,000 tons of coal per annum.

The sea-wall, which was the most important part of the work, was started first, the idea being to build this wall and use it as a coffer-dam to enable all the works inside to be done in the dry. The wall was constructed on a concrete bag-work foundation up to 2 ft. 6 in. above low water, and from that point to quay-level of block-work and mass concrete in situ. The width of the wall was originally 8 ft. at the top—*i.e.*, quay-level, with counterforts every 25 ft., and a backing of 20 ft. of rubble from the excavations. This, however, was not carried out, as no suitable rubble was found in the excavations, and the wall was built of concrete, 17 ft. thick. The height of the wall varied according to the level of the bottom. At the north end it is about 20 ft. high to quay-level, and at the south end the maximum height is 40 ft. Quay-level is 5 ft. above high water, and the parapet on the wall is 10 ft. above that level. For the first 400 yards from the south end the side wall of the entrance channel and the sea-wall run parallel, and they were built together, making the wall in all 27 ft. wide at quay-level.

Two and a half years were required for the building of the wall. During the first two years very bad weather was experienced and very little progress was made. Frequent storms and rough seas delayed the work, and often washed away the temporary stagings and roads. Difficulty was also experienced in shifting the sand overlying the rock. This was in some cases as much as 11 ft. deep, and all of it had to be removed before the bag-work could be put in.

The concrete bag-work was all put in position by divers. Goliath cranes were employed for building the wall at the north or shallow end, and derrick cranes at the south end.

After two bad years a very fine summer in 1910 enabled the wall to be finished, more than 800 lineal yds. being constructed in six months.

The wall was closed in October, 1910, and after the parapet was erected and a bank of soft material was put in along the inside, the water was pumped out, and the wall was found to be practically watertight.

Along the front of the old breakwater and the south end of the old sea-wall 50-ton concrete blocks had been thrown in pell-mell to break up the seas and protect the walls. Some of these had to be moved to allow the new wall to be built, and were then replaced outside the new wall. After the completion of the new sea-wall more of these blocks were made and placed along the front where the wall is on a curve concave to the sea, so that now the wall is protected in this way practically from the south end to opposite the dock gates.

The gate entrance and the excavation in the dock and the dock walls were then immediately proceeded with. The gate entrance was built of concrete, the side walls being faced with freestone. The hollow quoins, roller-paths, and sill-face were granite. In eight months this work was all completed and ready for the erection of the gates.

The excavation in the dock, amounting to 570,000 cu. yds., and the dock-walls, which were built of concrete with granite copes and were nearly 6,000 ft. long, were all finished in ten months. The excavations throughout the works were in soft unformed rock, but most of it required blasting before being lifted by steam-navvies.

Once the dock was finished, a concrete coffer-dam of blocks was built across the gate entrance to keep it dry while the water was allowed to rise in the dock.

When the work in the dock was well advanced the entrance-channel was started. A coffer-dam was built at the outer end and one in the centre of the channel to enable all the work to be carried out in the dry.

The principal work in the channel, besides the excavation, was the underpinning and completion of the east side wall, which had been built only from ground-level along with the sea-wall, and the erection of the greenheart timber jetty along the whole of the west side, and on that part of the sea side beween the end of the side wall and the dock gates. The sides of the channel were taken out to a slope of $1\frac{1}{2}$ to 1, and the jetties were erected on the slopes. The old sea-wall across the line of the new channel was removed, the old concrete being used for forming a paved channel at the back of the sea-wall, to carry off any water coming over the sea-wall during storms. The fairway outside the new entrance was deepened by dredging.

The contract for the hydraulic hoists, pipes, accumulators, and dock gates was carried out by Messrs. Sir W. G. Armstrong, Whitworth and Co. A large hydraulic-power and electric-light station was erected at the north-west corner of the dock.

The main access to the dock for coal is from the north, and a yard capable of storing 30,000 tons of coal has been laid down about one mile from the dock. The coal coming from the west has to be brought to the dock and shipped at a higher level, and alterations to the existing docks and the railway lines round them will have to be made when the new dock is open for traffic.

The total cost of the works will be about £700,000.

Messrs. Blyth and Westland were the Engineers for the works, and the author acted as Resident Engineer.

ALTERATIONS AND IMPROVEMENTS OF THE PORT TALBOT DOCKS AND RAILWAYS DURING THE LAST DECADE.

By WILLIAM CLEAVER, M.Inst.C.E.

The author points out that experience of the maintenance and working of large engineering undertakings sometimes indicates that certain details in the original designs have proved to be not altogether suited to their purpose, and that the modifications found to be necessary would often afford valuable information if published. He therefore submits an account of the more important alterations and improvements carried out under his supervision during the last ten or twelve years at the Port Talbot Docks and railway.

After giving a short history of the undertaking, he proceeds to describe in detail the channel training works, sand suction dredging, the design of suction nozzles, floating discharge pipes, pontoons, etc.

Channel maintenance by means of sluicing is dealt with, and in connection with this the fallacy of river-scour is explained. Details are given of a reinforced concrete dam or groyne built in the river to eliminate the very scour which was originally expected to prove beneficial, and also the results of the erection of the dam, in reclaiming the river-bed, restoring bridge pier foundations, reducing dredging, etc.

Improved dock moorings, designed by the author, are described, and also the strengthening generally of existing moorings. These moorings designed by the author consisted of a timber pile and concrete mooring. A number of these were constructed and were found satisfactory both as regards cost and efficiency.

Mention is made of other concrete and reinforced concrete structures, which included a new Power-house in which a good deal of concrete was employed. Further, a new coal tip was constructed entirely in reinforced concrete, as also a new wharf nearly 1,000 ft. long. The author, in the course of his paper, mentioned that the Port Talbot Railway and Dock Company were among the first in this country to adopt reinforced concrete for dock structures, and he went on to explain his real reason for its adoption, which, briefly, was the following :—

About seven or eight years ago a quantity of refuse oil was allowed to escape from a vessel about to leave the dock and became ignited. A film of thick oil extended over a large area, and the heat was so intense that some slag pitching was fused. Had this happened in the neighbourhood of the timber jetties, a very serious accident might have occurred. Whilst pointing out that extreme care was needed in the use of reinforced concrete, both in the use of the material and supervision of workmanship, he was of opinion that its adoption was fully justified for dock work, and that it was even more fire resisting than steel.

The paper then describes the present methods of dredging in dock, and the deposition of the spoil for reclamation purposes, and gives particulars of other methods tried previously with the same end in view.

Details are given of the laying, by diver, of what is believed to be the first extensive submerged high-pressure hydraulic mains and returns in this country. The new electric and hydraulic power-station is described. An account is given of the damage done to the breakwaters, etc., by the great storm of December 16th, 1910, and the steps taken to repair the work and to prevent similar damage in future are described.

Reasons are given for the provision of new lock-gates, and the necessity for having additional sluices arranged in them, and a new method of fixing fender-chains for the protection of the gates from collision is also described.

The author gives reasons for his view that the usual aprons in front of sluice-culverts, etc., are inadequate, and also refers to the necessity of substantial dock drains.

The paper concludes with particulars of the methods adopted by the author for the prevention of subsidences in railway embankments, and also of special repairs carried out on a brick arch viaduct without interfering with the traffic.

An addendum to the paper describes the use of ammonal explosive with the Cordeaux fuse in the recent demolition of the old engine-house.

THE SALFORD TECHNICAL AND ENGINEERING ASSOCIATION.

REINFORCED CONCRETE CONSTRUCTION.

By HAROLD J. F. GOURLEY, B.Eng. ; Stud.Inst.C.E.

The following is an abstract of a Paper read before the Salford Technical and Engineering Association, and should be of particular interest in showing what is being done by the educational centres of the northern provinces to promote a thorough knowledge of concrete and reinforced concrete:—

By way of introduction it was pointed out that concrete increased in strength with age, was strong in compression and weak in tension, had valuable fire-resisting properties, and was watertight. Steel, on the other hand, was equally strong in tension and compression, but was liable to corrosion unless frequently and thoroughly painted; it could not be relied upon, either, to resist fire. The combination of concrete and steel gave a material which was economical as regards cost, with which the maintenance was practically nil, and in which the weakness of concrete on the tension side was overcome by using steel so surrounded by concrete as to be effectually protected against corrosion and fire.

For many years reinforced concrete was regarded with the suspicion attaching to any new material, but now its merits were widely appreciated and its use extended to all kinds of construction.

Taking beam design as embodying the general principles underlying the application of reinforced concrete, it was pointed out that the usual beam theory, implying linear distribution of stress and strain, held for considerably beyond the stresses allowed. If the steel is to take its share of the load it must act with the concrete—that is to say, the union between the two must be such that there is no possibility of slipping taking place. The conditions under which experiments on this point have been carried out leave much to be desired, but it is fairly clear that the adhesion stress allowed in practice—60 to 100 lb. per sq. in.—gives an ample factor of safety.

Turning to the reinforcement, it was stated that plain bars are not always used. Plain bars were cheaper, more readily obtained, and seemingly developed quite as much adhesion as is called for. Deformed bars are said to offer a greater resistance to slipping, but in many cases there is a tendency to burst the concrete; they offer greater facilities for securely fixing the shear reinforcement, and for this reason are preferred by some.

The comparative advantages of hard (elastic limit 60,000 lb. per sq. in.) and soft (40,000) steel were briefly considered, and it appeared that there was little to choose between the two kinds.

In considering the shear resistance of the reinforced beam some assumed that the concrete could carry up to 60 lb. per sq. in., and if this proved insufficient metal was provided in the form of inclined bars attached to the main reinforcement. Others ignored the shear resistance of the concrete and provided sufficient metal in the form of inclined bars or vertical stirrups to take all or most of the shear.

Whatever form of reinforcement was adopted, care should be taken that any loose scale, rust, or oil was removed, though a slight rust film was not objectionable.

The reason for the formation of hair cracks on the under side of beams was given, and whilst in general such cracks were harmless in exposed situations, they might allow of the gradual exposure of the reinforcement.

Because the co-efficients of thermal expansion of concrete and steel were so nearly equal, and concrete itself was a fair insulator, no ill effects in the combination were likely to follow variations in temperature.

The variations in the volume of concrete with differences of moisture content for long periods after setting were alluded to, and it was suggested that by adding a small amount of mineral oil to the concrete such variations could be eliminated. Further research was required to give a better idea of the effect of adding oil, alum, and other chemicals to concrete used in reinforced work, particularly as regards the setting and strength of the concrete, and their action on the reinforcing metal and on the adhesion between concrete and metal.

The most suitable aggregate for almost all cases was undoubtedly washed granite drippings, when they could be obtained at a reasonable price. For harbour works ballast was frequently used, and elsewhere broken bricks and stone had been employed. Coke breeze or other material containing sulphur was to be avoided at all costs.

The stones should be varied in size, and the maximum allowable sizes may be taken at ¾ in. for small beams and slabs, and up to 1½ in. or 2 in. for larger members.

The proportions are usually such as not to be leaner than four parts of stone, two parts of sand, and one part of cement by volume, and where watertightness is required, 3, 1½ and 1 would be the probable ratios. The point being always to get a concrete of the maximum density, for this has the greatest strength, the greatest fire-resistance, and is most impervious.

The minimum amount of cover necessary for efficient protection against fire depends on the probable duration of the fire, but in an average case may be taken at 2 in. in main girders and columns, 1½ in. in beams, and 1 in. in slabs. Cases were known in which the cover given was very much less than the figures mentioned, and whilst the practice gave a slightly cheaper building, it was likely in cases of failure to reflect detrimentally on the otherwise undoubted ability of reinforced concrete to resist fire.

In water and sewage works the cover depended on the proportions of the concrete and the size of the stone, but should be at least 1 in.

Concrete, when it has taken its initial set, was under the best conditions when in contact with fresh water. It had been found that sea water in most situations had no effect on concrete, nor had sewage of average strength; but strong sewage flowing in ill-ventilated sewers had caused trouble, particularly at the general fluid level.

Vegetable and animal oils appeared to have a deleterious effect on concrete, particularly when long in contact with it.

Since it appeared that the failure of the concrete in such cases was due to the expansion of certain chemical constituents of the cement, brought about by the absorption of acids from the surrounding liquid, it pointed to the desirability of getting a concrete of the maximum density for that class of work.

The lecturer proceeded to show a large number of slides illustrating the many and varied uses to which reinforced concrete had been put, and whilst generally commending its employment, mentioned some points which had been criticised and where there was room for improvement.

A discussion followed, to which the lecturer replied at some length.

NEW WORKS IN CONCRETE

AT HOME AND ABROAD.

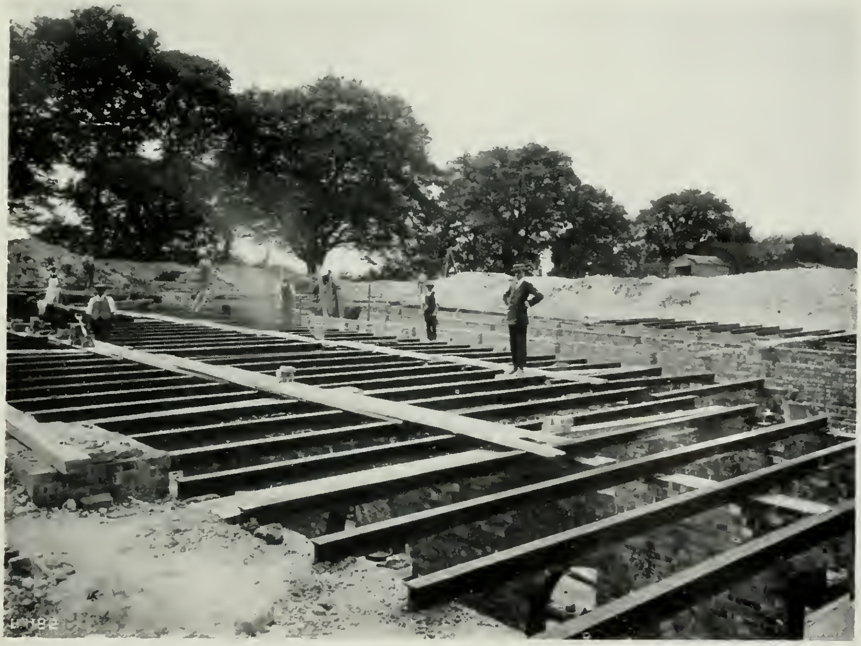
Under this heading reliable information will be presented of new works in course of construction or completed, and the examples selected will be from all parts of the world. It is not the intention to describe these works in detail, but rather to indicate their existence and illustrate their primary features, at the most explaining the idea which served as a basis for the design.—ED.

CONCRETE RESERVOIR AT LETCHWORTH GARDEN CITY.

THE needs of the town have increased so rapidly of late that it was decided by the First Garden City, Ltd., to enlarge the storage of their reservoir from 250,000 gallons to 750,000. Their engineer was accordingly instructed to prepare plans for the construction of a new reservoir of 500,000 gallons capacity. This was commenced in December, 1911, and finished by the end of July, 1912.

The ground in which it is built consists of hard chalk, and the top of the reservoir is, on an average, just at ground level. Consequently all the walls but one have a solid backing, and this has been taken into account in designing same.

With the exception of the roof (which is constructed in reinforced concrete) and six longitudinal brick division-walls for carrying the roof, the reservoir is built entirely of 6 to 1 concrete. The dimensions are 102 ft. by 90 ft. by 12 ft. deep, as shown in the section. The sides commence from the floor by a 45° slope of concrete 6 in. thick;

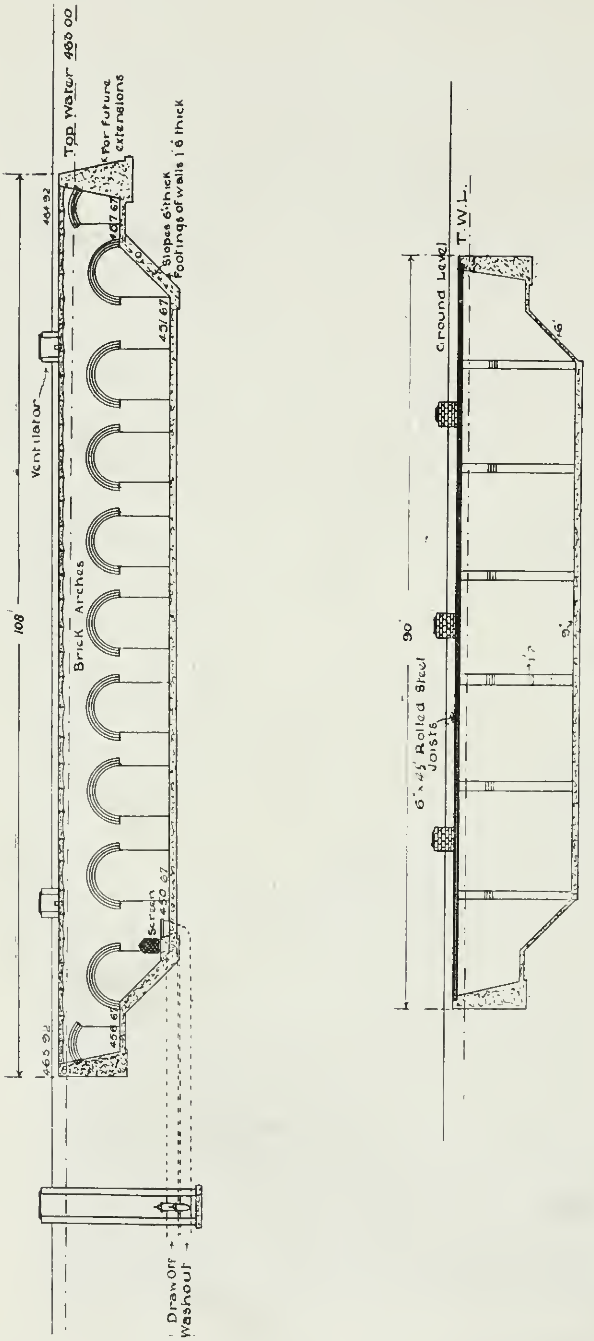


Showing the Reinforcing Joists to Roof.
CONCRETE RESERVOIR AT LETCHWORTH.

then there is a level berm, on the east and west sides 3 ft. 9 in. wide, and on the north and south sides 6 ft. wide; after this there is a retaining wall 8 ft. high, and battering on the west, east, and south sides of the reservoir from 3 ft. at its base to 1 ft. 6 in. at its top. One side of this wall is vertical. At the north end of the reservoir, and for a short distance along the west side, the wall—as shown in the section—has two battered faces, diminishing from 4 ft. 6 in. at the base to 1 ft. 6 in. at the top. This wall is so built that it may form the dividing wall between this and a future reservoir, and is designed to stand without any earth backing. The roof of the reservoir is constructed of 4 to 1 concrete jack-arches, 6 in. thick at the crown and 8 in. thick at the springing, reinforced by 6 in. by 4½ in. rolled steel joists. These joists are carried upon the six before-mentioned division walls. These latter walls are each composed of a row of nine semi-circular arches of 3 ft. radius, and two small arches, one over the berm at each end of the wall.

The interior is lined with natural rock asphalte ¾ in. thick, the roof also having a ½ in. covering of the same material.

The whole reservoir has a slope of 1 ft. in its length, thus enabling any small deposit to be removed easily by means of a washout pipe when cleaning is done. The inlet, overflow, draw-off, and washout pipes all leave the lower end of the reservoir and, with the exception of the overflow, are 12-in. pipes. The overflow is a 14-in. pipe.



Sections.

Reinforced Concrete Reservoir at Letchworth.

Two boreholes one mile away constitute the sources of supply, and, pumping against a total head of 315 ft., convey to the reservoir, and thence to the town, a plentiful supply of excellent water.

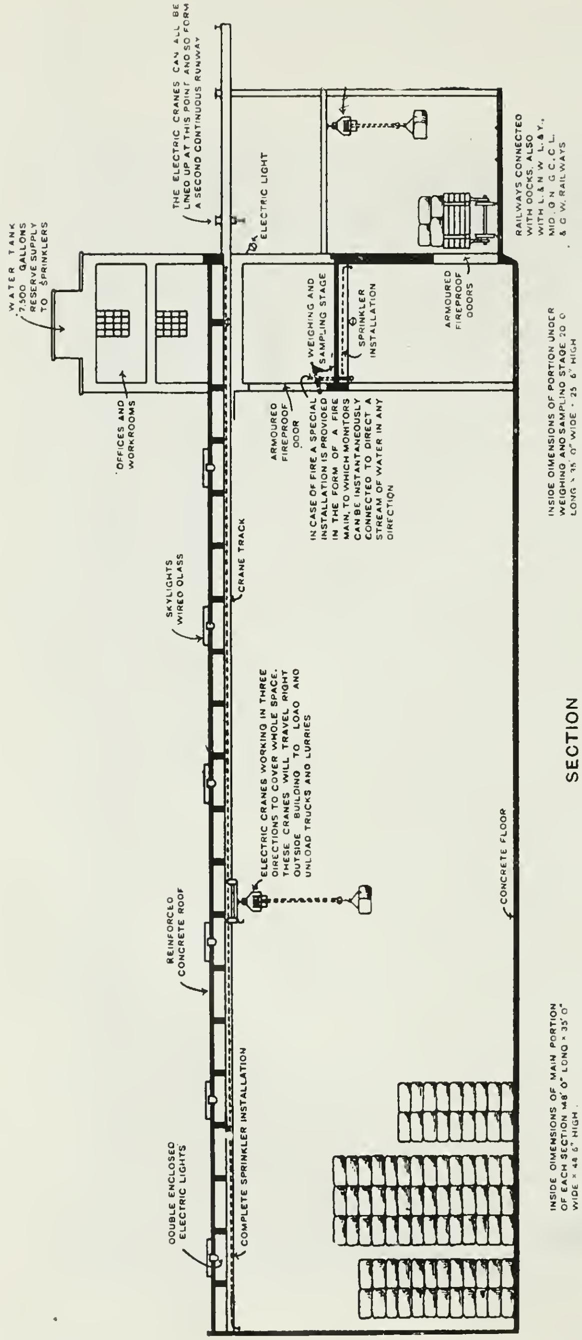
We are indebted to Mr. A. W. E. Bullmore, A.M.Inst.C.E., engineer to the First Garden City, Ltd., and under whose personal supervision the work was carried out, for our illustrations and the above particulars. The contractor was Mr. C. F. Ball of Letchworth.

Showing the Reinforced Concrete Beams ready to be lifted into position.

REINFORCED CONCRETE COTTON WAREHOUSES.

Photographer]

[R. Banks, Manchester.


WATER TANK 7,500 GALLONS RESERVE SUPPLY TO SPRINKLERS
THE ELECTRIC CRANES CAN ALL BE LINED UP AT THIS POINT AND SO FORM A SECOND CONTINUOUS RUNWAY
ELECTRIC LIGHT
OFFICES AND WORKROOMS
SKYLIGHTS WIRED GLASS
CRANE TRACK
REINFORCED CONCRETE ROOF
DOUBLE ENCLOSED ELECTRIC LIGHTS
COMPLETE SPRINKLER INSTALLATION
ELECTRIC CRANES WORKING IN THREE DIRECTIONS TO COVER WHOLE SPACE. THESE CRANES WILL TRAVEL RIGHT OUTSIDE BUILDING TO LOAD AND UNLOAD TRUCKS AND LURRIES
ARMOURED FIREPROOF DOOR
IN CASE OF FIRE A SPECIAL INSTALLATION IS PROVIDED IN THE FORM OF A FIRE MAIN, TO WHICH MONITORS CAN BE INSTANTANEOUSLY CONNECTED TO DIRECT A STREAM OF WATER IN ANY DIRECTION
WEIGHING AND SAMPLING STAGE
SPRINKLER INSTALLATION
ARMOURED FIREPROOF DOORS
RAILWAYS CONNECTED WITH DOCKS, ALSO WITH L. & N. W. L. & Y., MID. G. N. G. C. C. L. & G. W. RAILWAYS
CONCRETE FLOOR
INSIDE DIMENSIONS OF PORTION UNDER WEIGHING AND SAMPLING STAGE 20' 0" LONG × 35' 0" WIDE × 25' 6" HIGH
INSIDE DIMENSIONS OF MAIN PORTION OF EACH SECTION 148' 0" LONG × 35' 0" WIDE × 44' 6" HIGH.

SECTION

Reinforced Concrete Cotton Warehouses.

REINFORCED CONCRETE COTTON WAREHOUSES.

Owing to the lack of storage accommodation at the Manchester Docks, the Trafford Park Estates, Ltd., have recently erected a group of nine warehouses or "cotton safes" extending along to Bridgewater Canal, Manchester, with easy connection to the main docks. The nine warehouses are so planned as to be all at ground level, and, with the exception of the skylights, have been constructed entirely in reinforced concrete. The buildings measure 165 ft. long by 35 ft. wide, and they are about 45 ft. in height. The interior cubical capacity of the nine warehouses is 2,200,000 cu. ft., equal to the space occupied by about 78,500 bales of American cotton. But it is estimated that only about 50,000 bales will be stored, as much of the space is to be occupied with overhead travelling cranes and for the purposes of marking and numbering the cotton before storage.

The warehouses are divided from one another by hollow walls. The roof also is of reinforced concrete carried upon reinforced stanchions which form part of the wall construction.

Each of the nine sections is fitted with three motor cranes, which will enable the bales of cotton to be transferred from barge to warehouse in a minimum space of time.

The warehouses were erected according to a project of Mr. Marshall Stevens, Managing Director of the Trafford Park Estates, Ltd., to which company we are also indebted for our particulars and the illustration showing the arrangement of the safes.

REINFORCED CONCRETE WORK IN SWEDEN.

Concrete and reinforced concrete are gaining considerable ground in Sweden for structural works of every description, and we give herewith two recent examples where reinforced concrete has been employed to great advantage.

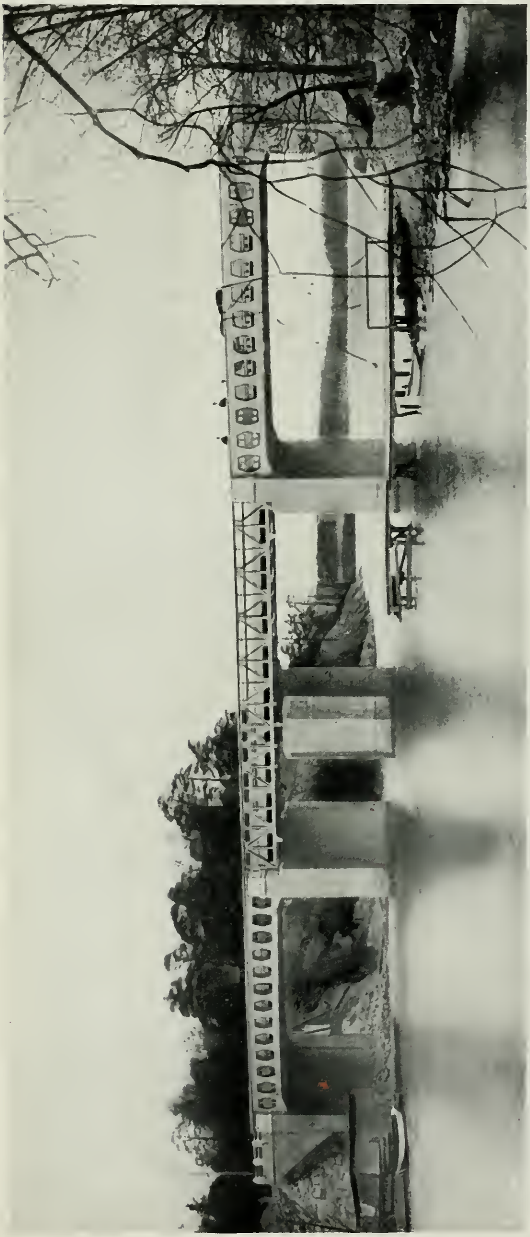


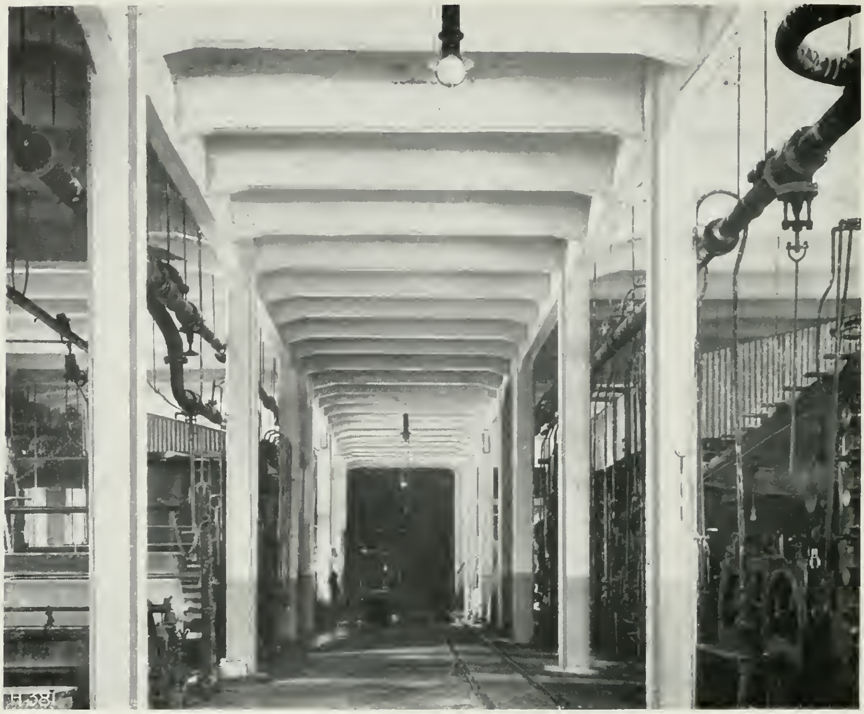
Fig. 1. A Reinforced Concrete Bridge near Kungsholmen, Stockholm.

A Bridge near Kungsholmen, Stockholm.—*Fig.* 1 is an illustration of a bridge near Stockholm. The reinforced concrete spans are built on the Verendee system. The free width of span is about 52 ft. 6 in. The height of the beams is 6 ft. 6 in. The bridge was constructed for a movable load of about 2 cwt. per sq. ft.

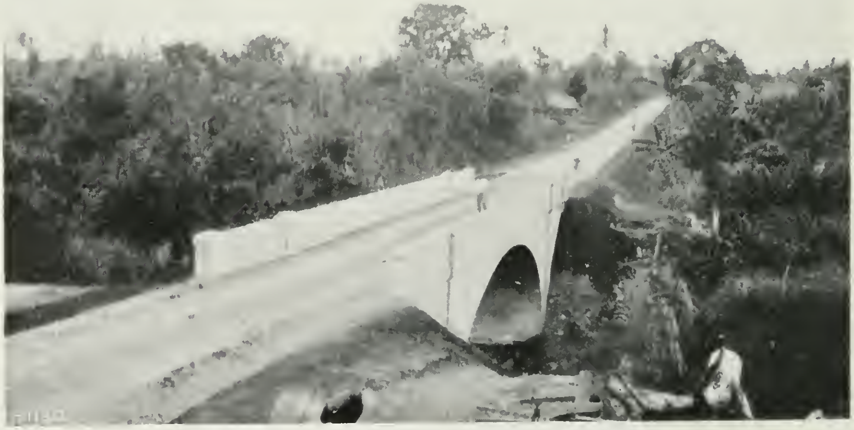
Paper Mills at Sjursfors, Sweden.—Our second illustration, of some paper mills, is of particular interest, and would seem to open up a further field for the use of reinforced concrete, inasmuch as the wooden beams and pillars were replaced by the new material without any stoppage of the working of the mills. The forms for the casting of the concrete were erected on the existing wooden beams, and those for the concrete pillars close to the old wooden pillars. The casting took place while the mills were in operation. When the concrete had settled sufficiently the wooden pillars, beams, and the forms for the concrete were removed. These mills are 164 ft. long by about 62 ft. high.

REINFORCED CONCRETE IN THE PHILIPPINE ISLANDS.

A Concrete Bridge.—Our last illustration shows a bridge of 65 ft. span which has been erected in Leyte Province, Philippine Islands, at a cost of about 4,500 dollars.



Reinforced Concrete Paper Mills at Sjursfors.



Reinforced Concrete Bridge at Leyte, Philippine Islands.

POPULAR USES.

Under this heading it is proposed from time to time to present particulars of the more popular uses to which concrete and reinforced concrete can be put, as, for instance, in the construction of houses, cottages and farm buildings.—ED.

CONCRETE FARM BUILDINGS.

THE question of a cheap building material for farm purposes has long occupied the attention of those interested in work of this kind. Concrete is not only economical, but structures erected in this material are also healthy owing to the fact that they can be cleansed frequently and easily, and thus the likelihood of disease among the farm cattle is considerably minimised.

Our illustrations show some farm buildings erected entirely of concrete and concrete blocks on the estate of Wayford Tenants, Ltd., Norfolk.

Fig. 1 covers a ground area of 28 ft. long by 14 ft. wide, and shows two cow-stalls, a fodder-room, and two pigsties. The two cow-stalls are 12 ft. by 7 ft. 6 in.; the fodder-room is 14 ft. by 6 ft. The divisions in the pigsties are 7 ft. by 7 ft., with a courtyard to each 7 ft. by 6 ft.

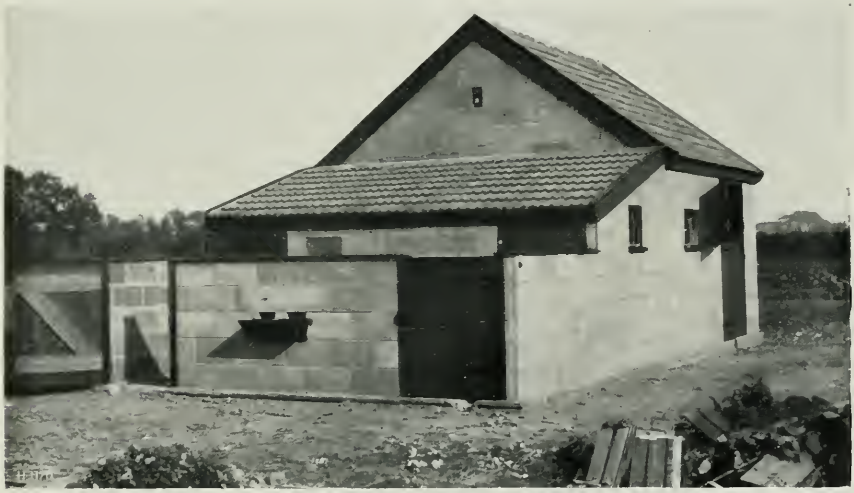


FIG 1. COWSHED, FODDER-ROOM AND DOUBLE PIGSTY.

In the courtyard there is a trough made entirely of concrete with a shoot from the outside wall to the trough, so as to avoid going into the pigs' courtyard at feeding time. From the fodder-room there is a feeding passage at the head of the cow-stalls; the manger is of reinforced concrete, and the Danish principle of having it low has been adopted. The walls are all built of concrete 4½ in. thick, the size of each block being 18 in. by 9 in. by 4½ in. The roof is of concrete tiles (glazed red). These tiles form a very warm and absolutely waterproof roof.

The building is fitted with a concrete tank sunk in the ground, to which all the liquid manure is drained. This is a very essential point to the small farmer who keeps very little stock. The whole cost of this building was £31, and it is interesting to note that it was erected in fourteen days with the aid only of one blocklayer and his labourer.

Figs. 2 and 3 show a double pigsty, the materials used in construction being the same.

Fig. 4 shows a concrete block building in course of construction. This is on a somewhat larger scale, there being a harness-room 12 ft. by 6 ft., a stable 12 ft. by 12 ft., a cow-shed 12 ft. by 15 ft., and also a double pigsty.

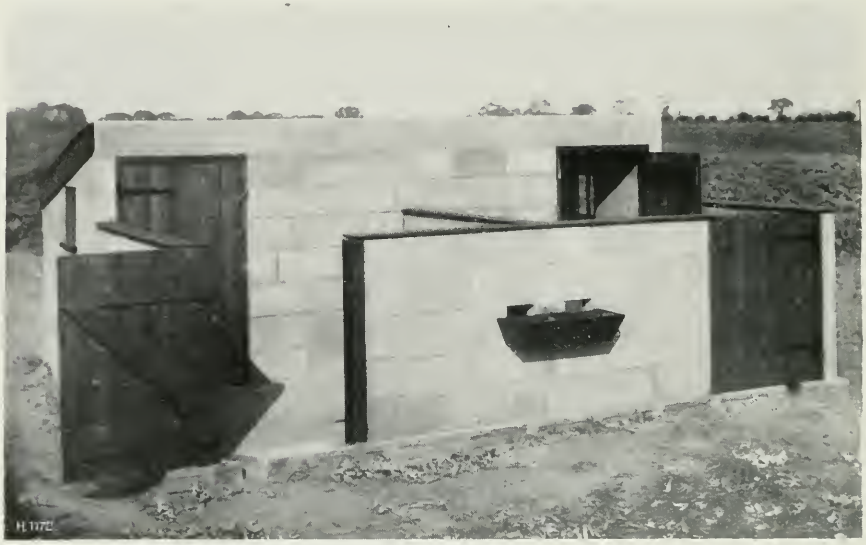


Fig. 2. Front View of Double Pigsty.

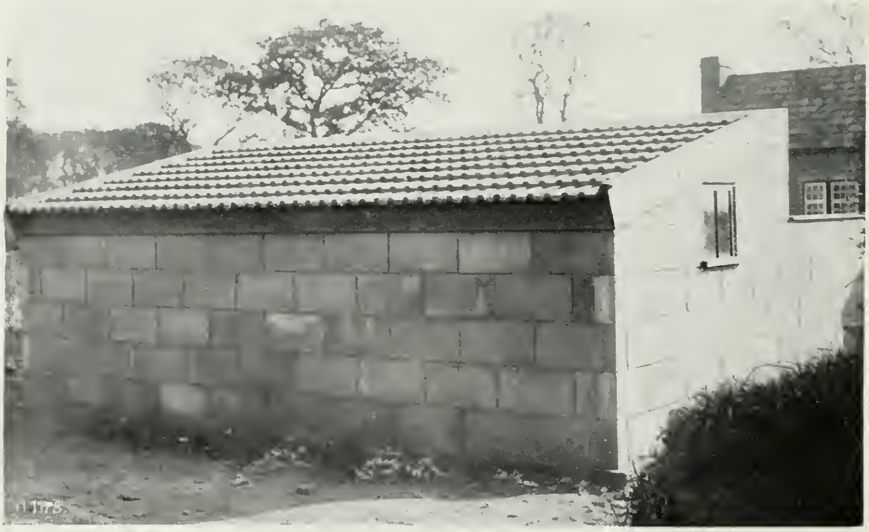


Fig. 3. Back View of Double Pigsty.

The blocks and tiles were made on the estate on machines supplied by Messrs. R. H. Baumgarten.

The buildings were erected under the supervision of Mr. B. F. Hartley, the works manager.

We are informed that the Departmental Committee on Rural Housing appointed by the Board of Agriculture visited Wayford recently to inspect the cottages illustrated in our October issue, and expressed their satisfaction with the buildings.

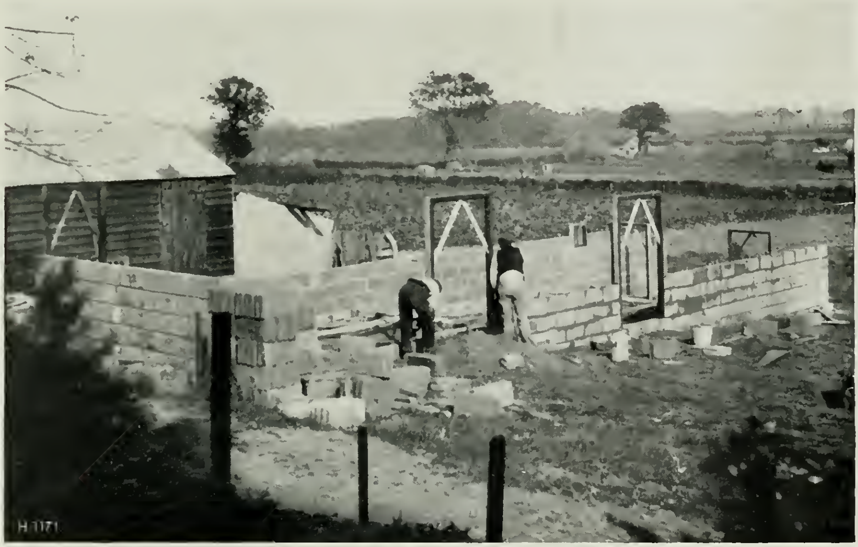


Fig. 4. A Farm Building in course of construction.

NEW BOOKS
AT HOME AND ABROAD.

A short summary of some of the leading books which have appeared during the last few months.

"Reinforced Concrete Construction." Advanced Course by M. T. Cantell.

London: E. & F. V. Spon, Ltd., 57 Haymarket. 240 pp. + xvi. and 7 plates. 12/6 net.

Contents. — Additional Formulæ — Columns — Piles — Walls—Retaining Walls—Strong Rooms—Stairs—Roof Construction—Construction of Water Tanks—Water Towers—Reservoirs or Tanks below Ground Level—Swimming Baths—Bunkers, Bins, Silos—Water Mains, Sewers and Conduits—Circular Tanks or Reservoirs—Arches and Bridges — Circular Hollow Columns and Tall Chimneys—Buildings and Foundations.

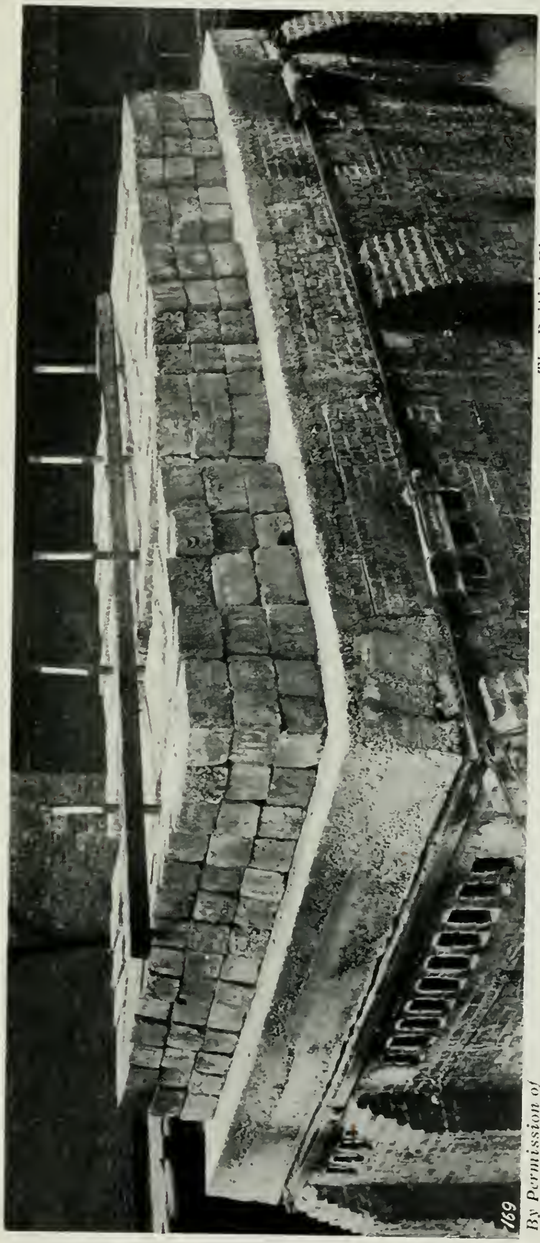
This volume forms the continuation of the elementary course by the same author which was recently reviewed in these columns, and a knowledge of the principles and elementary formulæ is assumed. Like the elementary course, the volume has the merit that the derivation and construction of the various formulæ are clearly shown, and in this respect it has an advantage over many works on the subject where the method is not adopted. Although a fairly wide scope is covered by the author, there are certainly some chapters which are quite inadequate and do not deal with the subject in sufficient detail to be of any material value. As an instance of this, it will be found that only two pages are devoted to stairs and only about five pages to piles, and in the latter case no less than three of the pages are given up to photographic views. It is to be regretted that the whole subject was not dealt with more consistently, as this would have greatly enhanced the value of the work as a textbook. The matter devoted to retaining walls, bunkers and silos, and arches is very comprehensive and has been well written, and the theoretical portion should be easily understood by any reader. The whole of the plates are illustrative of arch design, and they are generally superior to the other diagrams given in connection with the theoretical portions of the book. A great number of the illustrations are photographs and drawings reproduced by permission of specialists, and the references are so numerous that they render the work almost an advertisement pamphlet. It would have been preferable if this had been avoided and a general acknowledgment given at the commencement of the volume. It is also a great advantage to the student if some working drawings are given to accompany the photographic views, as these will render the example more instructive and intelligible, and it is preferable to give a few complete examples than many incomplete ones. It must not be considered that the volume is a poor one in view of these criticisms, as we would wish to add a word of praise also. It should prove of much value to the reader who has already been through the elementary course, and it is a volume well worth the outlay entailed by its purchase, as it is written in a manner that should enable the student to follow the author and really understand the subject he is studying.

"Industrial Chemistry—A Manual for the Student and the Manufacturer." Edited by Allen Rogers and Alfred B. Aubert.

London: Constable & Co. Ltd., 1912. 24/- net.

In this volume, of American origin, the attempt is made to present a survey of the whole field of chemical manufactures in the form of a collection of articles, in each of which an author deals with some branch of industry with which he has a special acquaintance. Evidently the description of any one branch, confined as it is within exceedingly narrow limits, cannot convey anything new to those engaged in that industry. Thus the metallurgy of iron and steel is described in eighteen pages, including the bibliography; that of glass in sixteen pages, and so on. This being premised, it may be said at once that the individual articles are remarkably well done, and that it has been found possible to compress much useful information into the small space. Only American practice is dealt with, and the machinery, etc., described is almost exclusively of American manufacture.

The account of materials used in the chemical industries generally includes cement and concrete as structural materials and for the construction of vats, but no allusion is made to the use of Portland cement clinker as a refractory lining, or to that of cement grout as a rust-preventing coating for steel. Reinforced concrete does not appear to be mentioned.



By Permission of The British Fire Prevention Committee.

View of Floor at Conclusion of Fire Test.

The article on "Lime, Cement, and Plaster," by Mr. Richard K. Meade, occupies twenty-two pages, of which about ten pages are devoted to Portland cement. Within these very narrow limits a very clear account is given of the manufacture of the cement by the dry and the wet processes, especially in the Lehigh district. No attempt is made to discuss the chemistry of the process, and although the addition of gypsum is described, there is no reference to any other method of regulating the setting time, or of the properties of cement generally. Cements made from blast-furnace slag, whether mixed with lime or with Portland cement clinker, are not regarded by the author as Portland cements, but are conveniently classed as "Puzzolan" cements. No bibliography is given in this case, each author having been left to supply such an adjunct to his article or not; hence the description is more likely to be of interest to the general reader than to the student. If read, however, with a due sense of proportion, and with reference to other and more complete works, the book should be of interest to many seeking for a readable account of modern chemical industries without demanding much detail. The printing and illustrations are excellent.

"Fire Tests with Floors." A Floor of Reinforced Concrete Reinforced with "Triangle Mesh" Reinforcement and Presented for Test by the United States Steel Products Co.

London : Published at the Offices of the British Fire Prevention Committee ("Red Book" No. 169), 8 Waterloo Place, Pall Mall, S.W. 8vo. 3/6.

Those interested in the fire resistance of reinforced concrete should find this report of considerable interest. The floor was put forward for test under the Committee's Standard Classification, "Full Protection" (Class B). As will be seen from the report, this classification was obtained. The floor was under fire for three hours, and during that period was subject to temperatures up to 1,800° F. Water was applied for five minutes at the conclusion of the test. Photographs were taken before and during the test, as also at its conclusion, and these illustrations are included in the book, which in addition contains tables, diagrams, and charts, which enable the reader to follow the course of the test with ease. Further, the metric equivalents of all dimensions, etc., are given. We would add that the area of the test piece under observation was 334 ft. super., divided into three equal reinforced concrete bays, supported by four rolled steel beams, which latter had a span of 15 ft.

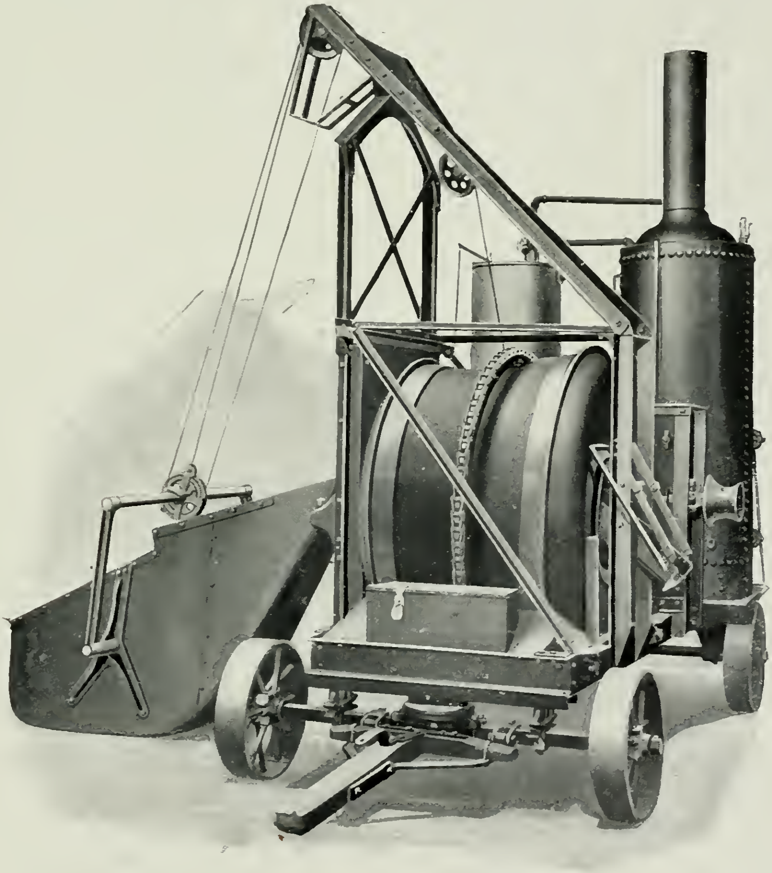
There is an appendix to this report dealing with a load test applied to the same floor, and, according to the report, the object of this load test was "to record the result of applying a load of about 5 cwt. to the sq. ft. (2,734 kg. per □ m.) to the centre bay of the floor of reinforced concrete." According to the summary of the test, "the maximum deflection registered in the centre of the bay after loading the bay to the extent of 5 cwt. = 560 lb. per ft. super. was 4·10 in." Our illustration shows a view of the floor at the conclusion of the fire test.

INDUSTRIAL NOTES.

These pages have been reserved for the presentation of articles and notes on proprietary materials or systems of construction put forward by firms interested in their application. With the advent of methods of construction requiring considerable skill in design and supervision, many firms nowadays command the services of specialists whose views merit most careful attention. In these columns such views will often be presented in favcur of different specialities. They must be read as ex parte statements—with which this journal is in no way associated, either for or against—but we would commend them to our readers as arguments by parties who are as a rule thoroughly conversant with the particular industry with which they are associated.—ED.

CONCRETE MIXERS.

The main feature of the mixer illustrated herewith is that, in place of the more or less unsatisfactory and troublesome method of gear-wheel driving, the drum is driven from a sprocket-wheel on the driving shaft by means of a strong endless steel chain passing completely round the drum and working over sprocket teeth bolted in sections to the drum.



Chain Belt Concrete Mixer (with Power-Loader and Water Tank).

This elimination of gear-wheels is an important improvement, owing to the immense amount of wear such wheels are subjected to with heavy machinery of this kind and dealing with substances of such an abrasive nature as sand and cement. It is, of course, quite a simple matter to adjust a chain that has worn slack, but once a gear-wheel becomes worn it is practically useless.

Another feature in the above machine is the cast semi-steel drum made from a special mixture exceptionally hard, tough and durable. This cast drum takes the place of the ordinary drum made from riveted steel plates, and, being naturally considerably thicker, it should have a very much longer life, while there is no difficulty of rivets becoming loose and causing leakage. The interior surfaces of the drum are designed with liberal curves to prevent any clogging or building up of the material. It will also be noticed from the illustration that the drum is cast to a special *bulged* design. This bulge is intended both to increase the capacity and to form a pocket for the elevating buckets to ride in, and thus aids in filling and keeping the buckets full. The bulge greatly shortens the time in which a batch can be mixed and discharged as compared with a straight bottom drum.

The mixing blades are made of $\frac{1}{4}$-in. steel riveted to cast steel brackets bolted to the drum. The blades are raised to allow a free passage of water under them. The elevating buckets are made of high grade malleable iron, and are specially placed and shaped with a view to elevating the material with the greatest speed and facility.

The wide mixer frame is solidly constructed of heavy steel channels and angles, well braced and riveted together, to give a strength and rigidity so as to assure steadiness under all conditions of operation and prevent the necessity of blocking up, and the consequent waste of time and labour. The extra width of frame allows the engine and boiler to be placed side by side, so that the engine flywheel can be used for driving pumps, circular saws, etc., when desired.

The mixers are supplied in several sizes according to the capacity required. With the smaller machine, where a very large output is not required, the standard hopper is generally found to be the most useful means of loading the drum.

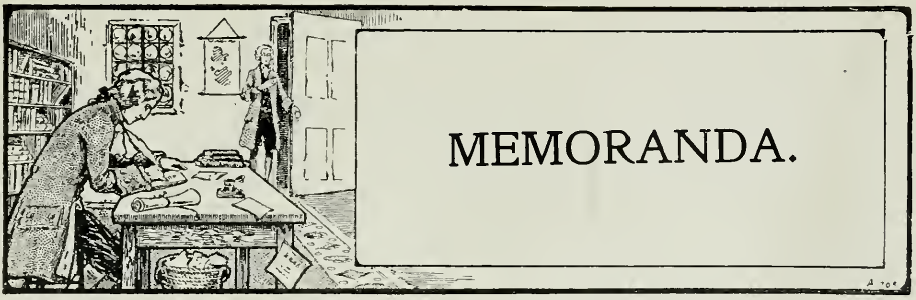
Batch hoppers are also supplied to hold the entire batch, and are very useful in cases where the aggregates can be conveniently charged from a higher level by means of wheel-barrows, etc.

Where a very large output is necessary the power-loading appliance, shown in the illustration, is very useful. The bucket or hopper is specially shaped with a view of the material being quickly discharged into the drum, without clogging or spilling. It is claimed that this power-loader greatly increases the daily output, and very considerably reduces the cost of mixing per cubic yard. The loader is filled while the previous batch is being mixed in the drum, so that the drum is practically always full and the mixing operation continuous.

There is also an automatic water tank with the view of ensuring an exact amount of water entering into each batch.

It may be added that the mixer is of the horizontal type, the drum being filled by the hopper while in its horizontal position, without the necessity of tilting.

The mixer is made by the Messrs. Bagshawe & Co., Ltd., of Dunstable, from whom all further particulars are obtainable.



MEMORANDA.

Memoranda and News Items are presented under this heading, with occasional editorial comment. Authentic news will be welcome.—ED.

The Institute of Sanitary Engineers.—By the courtesy of the Associated Portland Cement Manufacturers (1900), Ltd., some of the members of the Institute of Sanitary Engineers visited the Swanscombe Portland Cement Works at the end of October. Demonstrations for the testing and gauging of cement were given.

Barges of Reinforced Concrete.—It is stated that, owing to their cheaper cost, reinforced concrete barges are being employed on the Panama Canal. The ordinary flat-bottomed type is used, and their dimensions are 64 ft. by 24 ft. by 5 ft. depth. There are two longitudinal bulkheads, and lateral frames 10 ft. apart strengthen the construction.

Concrete Block Foundations.—In connection with the recent floods in Norfolk a new railway viaduct was built, and the foundations to this viaduct consist, says *The Times*, of concrete blocks measuring 90 ft. by 30 ft. by 6 ft. deep, sunk 5 ft. below the bed of the stream.

Concreting in Winter. — Concreting in winter at temperatures not exceeding 10 deg. to 15 deg. Fahr. was successfully carried on in the case of a large reinforced concrete warehouse at Delavan, Wis. The outer walls of the structure were of brick, and were built up to a height a little above each floor level before the concrete was poured. The concrete plant was in the basement, and the materials were heated by steam passing through a large coil of $1\frac{1}{2}$-in. pipe exhausting into the water barrels at the concrete mixer. The materials were dumped directly over the steam pipe, which was laid in the form of a continuous coil. It was considered that this arrangement afforded better circulation of steam heat than the use of parallel pipes and headers. At first exhaust steam was used in the coils, but later the full boiler pressure was employed, and it is stated that the latter method gave much better results. In this way the temperature of the concrete was kept between 50 deg. and 60 deg. After the concrete was poured it was covered with canvas, and salamanders were placed on the floor below, one salamander being employed to about 600 sq. ft. of floor space.—*The Engineer.*

Reinforced Concrete for Church Foundations.—The foundations for the tower of the new church at Berchem-lez-Anvers, estimated to cost 424,566 fr. (£16,982 12s.), are to be laid on an impervious concrete raft, or platform, at least 80 centimetres (2 ft. 8 in.) thick, reinforced by three rows of round bars 20 millimetres ($\frac{13}{16}$ in.) in diameter, and at most 20 centimetres ($7\frac{13}{16}$ in.) apart, crossing one another and interconnected. A floor (*radier*) of hard bricks set in cement—$1\frac{1}{4}$ ton of cement to a cubic metre (1·09 cubic yard) of sharp sand—is to extend under all the walls, as well as under the arches and columns.—*Contract Journal.*

The First Concrete Beater Tubs in the World.—It appears that the reinforced concrete beater tubs which are being built by the Aberthaw Construction Co., of Boston, in the new reinforced concrete buildings of the Milton Leatherboard Co., Milton, N.H., are the first beater tubs to be made of the modern structural material. Generally, beater tubs have been made of wood and lined properly. The proposed beater tubs are four in number, and have overall dimensions 26 ft. 8 in. long by 13 ft. 2 in. wide, making them larger than any wooden tubs which have heretofore been built. It is stated that the adoption of reinforced concrete for this purpose will have many decided advantages, and the results obtained with this material will be looked forward to with interest.

Waterproofing Concrete Floors.—Concrete floors, says the *American Roofer,* should never be treated or dressed until they are thoroughly dry and well seasoned, and the dressing should be applied before they have been subjected to traffic or wear. In some instances it is necessary to use an acid wash or some other neutralising medium before applying the dressing to concrete floors.

When the concrete is thoroughly dry and seasoned two coats of concrete floor dressing should be applied. A cement mortar veneer of $\frac{1}{2}$ in. thickness—and applying two coats liquid cement upon the surface, after the same is thoroughly dry—produces a sanitary and light radiating basement.

Concrete floors having exceedingly fine, close texture, floated to a dense surface, which carry at times a semi-gloss finish, should be treated as follows: The surface should be gone over lightly with a carborundum rubbing brick in order to establish a bonding surface that will enable the concrete floor dressing to knit close to the floor structure and practically become integral with it. This will prevent scalping and peeling of the floor dressing. The floors should, of course, be clean, thoroughly dry, and well seasoned before the first or priming coat of concrete floor dressing is applied. Transparent floor dressing of any colour desired may be used. It should be applied by means of a suitable brush, preferably a flat 5- or 6-in. white bristle brush. The dressing should be worked well into the pores of the concrete, spreading it well over the floor surface to develop a thin film. The temperature conditions during the periods of application and drying should not be under 70 degrees Fahrenheit to insure the best results. The second coat should not be applied until the first or priming coat is thoroughly dried, which under proper temperature conditions requires from two to three days. The second coat should be brushed out evenly and uniformly over the surface in the same manner as one would apply a finishing coat of floor varnish, and it should be given from four to six days for thorough hardening. Two coats develop a very satisfactory result, though three coats should be used where floors are submitted to unusual wear, such as floors that are used as passages, aisles, and runways, over which heavy trucking is done.

Concrete floors of exceedingly coarse texture should be cleaned, thoroughly dry, and well seasoned before the first or priming coat of floor dressing is applied. Either the transparent floor dressing or any colour desired may be used, and should be applied by means of a suitable brush, such as is described for its use on concrete floors of fine texture. The dressing should be applied in the same way as for fine close texture floors. The same temperature conditions also apply as for fine texture floors. After the first coat is thoroughly dry, all surface areas or spots which may show unusual absorption or penetration of floor dressing should be "touched up" or gone over with an additional coat of dressing, in order to prepare a uniform surface for the finish-coat. After the first coat is thoroughly dry, the finishing coat should be applied, brushing same out evenly and uniformly over the surface, as described in the specification for concrete floors of fine texture. The finishing coat should be allowed to harden from four to six days.

Concrete floors of irregular texture—by which expression is meant those floors which are made up of extremely fine and extremely coarse texture, laid irregularly, which is sometimes due to carelessness, but often due to unforeseen conditions—should be given a joint application of the specifications described for concrete floors of fine texture and concrete floors of coarse texture.

Concrete floors of normal, uniform texture should be clean, thoroughly dry, and well seasoned when the first or priming coat of concrete floor dressing is applied. In this instance, as well as others, the transparent floor dressing, or any other colour desired, may be used, and it should be applied by means of a brush and worked well into the pores of the concrete, spreading the surplus well over the floor surface to develop a thin film.

Floors which are laid in colours, such as terra-cotta, green, etc., or where inert colours are mixed throughout the cement veneer or wearing surface, after being thoroughly cleaned, dried and well seasoned, should be given two coats of transparent floor dressing, applied under the same conditions as is described for the normal or uniform texture.

Paper versus Cloth Sacks for Cement.—When asked his opinion as to whether the best results were secured by buying cement in cloth sacks or in paper sacks, Leonard C. Wason, president of the Aberthaw Construction Co., Boston, stated that the correct answer to such a question is at the present time a much mooted one among the users of Portland cement. He then went on to say that he thought the time would never come when either form of package is entirely eliminated, but that as time went on and the buyers of cement analysed more carefully the methods of the question the use of paper sacks would steadily increase. In bringing out the disadvantages of the cloth package as compared to the paper package, Mr. Wason mentioned that, although it appeared at first sight that the cloth bags are cheaper than the paper bags, owing to the fact that the former, though charged for in the first instance, can be returned to the manufacturer and their cost recovered—which, of course, is not the case with paper sacks—yet there are so many other advantages attaching to the latter which quite outweigh any apparent initial higher cost.

There is the expense for prepaying freight on the cloth bags. Further, mention must be made of the time, labour, and expense of collecting, counting, storing, keeping tally, packing, cartage, etc., of the cloth bags, which item is considerable.

Cloth bags require time to untie and time and care to empty thoroughly. Paper sacks require little time to untie, can be handled easily, cut with a hoe, broken across a box or barrel, and emptied instantly.

The use of cloth bags involves a loss of cement in transit by sifting through the bag in handling. No such loss is sustained with paper bags. Again, in the one case there is a possible loss in tearing, but the use of a good grade of Manilla rope paper sacks reduces this risk to a minimum.

There is a further loss in emptying the cloth bags. Labourers are often so careless as to leave from 0·5 to 1·5 per cent. adhering to the sides of the bags. Paper sacks empty easily and no cement adheres to the sides.

The user of cloth sacks is almost sure to suffer loss through theft and carelessness. A cotton sack is an attractive article to labourers, as it has many uses.

Moisture is absorbed and easily penetrates the cloth bag, damaging the cement and causing it to stick to the bag; there is no loss from ordinary dampness and, in fact, cement in paper bags frequently goes through short rainstorms without damaging the cement.

Inasmuch as there is a charge on the cloth bags, there is a further loss if these are not returned, or are returned damaged or in bad condition.

After careful analysis of these various points, the conclusion arrived at is that a paper package when made of stout, tough paper gives a service which is well worth the apparent higher initial cost.

TRADE NOTICES.

Messrs. Bellamy's Wharf, Rotherhithe.—The work at this wharf has been carried out with the object of increasing the available depth of the berth, and was fraught with considerable difficulty, as the routine of the wharf could not be interfered with or work delayed. For this reason also the necessary height for a piling ladder was not available. The various difficulties were, however, successfully overcome, and the following description of the work may prove of interest.

As shown in the illustration, the jetty is carried on steel cylinders 5 ft. 6 in. in diameter, the foundation being hard grey ballast.

A temporary structure was erected about 4 ft. in front of the jetty, composed of 12-in. by 12-in. piles framed and strutted with half timbers. This formed a fender for the ships, and afforded means for fixing the leads for the hammer, the leads being secured to the half timber cross-bearers by angle irons and bolts. By this means the monkey was reversed, enabling the line of steel sheet piling to be driven close up to the existing cylinders.

Driving was carried out by means of electric winches operating 30-cwt. hammers. Two complete sets of plant were employed, the work being commenced by driving a single sheet in the centre and working right and left from same. The progress was of necessity rather slow, owing to the difficulties to be overcome. The main object was, however, successfully achieved—viz., the whole of the driving work was carried

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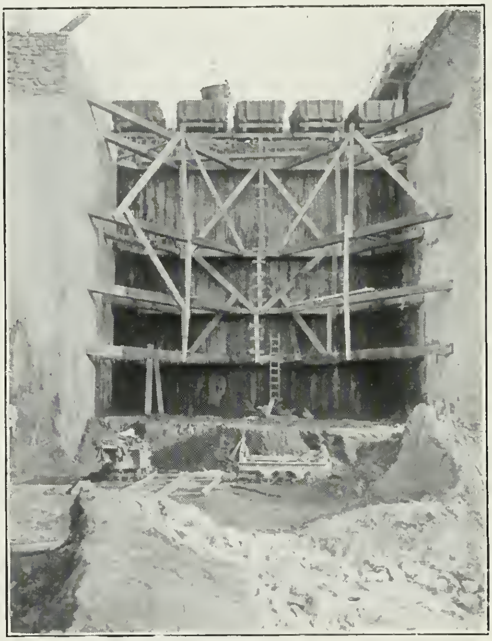

View of Dam at Brest Harbour, France.

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out without in any way disturbing the usual routine of the wharf, or interfering unduly with the loading of the ships.

The interlocking steel sheet piling having been driven along the front of the jetty, 2-in. tie rods 18 ft. long were secured to baulks of timber anchored down into the barge bed, hard core was then placed between the existing camp-sheeting and the steel sheet piling, and well rammed around the cylinders.

In front of each cylinder a 14-in. by 14-in. rubbing pile was driven, a 12-in. by 12-in. waling-piece was next placed in position level with the top of the work, and the steel sheeting bolted to same, the shackle bolt hole being utilised for this purpose.

A 6-in. rubbing piece was spiked on to the rubbing pile, bringing the work level with the face of the steel piling, and a good finish to the whole was obtained by means of a rounded capping secured to the top waling.

Steel Sheet Piling at Messrs. Bellamy's Wharf, Rotherhithe.

The contract has been carried out by Messrs. Chafen and Newman under the supervision of Mr. W. K. Myers, of Messrs. Bradshaw Brown and Co., Billiter Square Buildings, E.C., who, after carefully considering the respective merits of the various systems of steel sheet piling, specified Side Groove Steel Piling (Annison's patent) as being most suitable for the work.

Ironite.—Some interesting particulars have just reached us concerning a new patent waterproofing process for cement bricks and other porous substances, called "Ironite," and which is entirely of British make. It is said to consist of a fine mineral powder, and does not contain either oil, paraffin or asphalt. The makers deliver it packed in metal cans.

Its application is as follows: The "Ironite" is mixed with water to the consistency of ordinary whitewash and applied to the desired surface with a brush. It is claimed that the water will carry the material into any porous object and into any cavities that moisture will penetrate. According to the particulars before us, the value of "Ironite" as a waterproofer is due to the fact that when introduced into voids of concrete, etc., the "Ironite" particles oxidize and form both a chemical and mechanical union with the concrete. Once applied it is claimed the material is only removed with the utmost difficulty, and then only by taking a part of the concrete with it.

Ironite is particularly applicable to foundations, damp courses, tunnels, walls, reservoirs, pits, etc.

The process seems to have found great favour in the United States, and in this country it has been carefully investigated by Mr. Bertram Blount, F.I.C., who has submitted it to various tests, and whose report makes interesting reading.

Some tests have also been undertaken by Messrs. David Kirkaldy and Son. Full particulars of these tests, together with all details regarding the process, can be obtained from The Ironite Co., Ltd., whose agents are Messrs. S. Thornley, Mott and Vines, Ltd., 1, Victoria Street, S.W.

Ransome-verMehr Machinery Co. Ltd.—We are asked by Messrs. Ransome-verMehr Machinery Co., Ltd., to announce that they have removed to new offices and that their new address is as follows: Ransome-verMehr Machinery Co., Ltd., Brunswick House, Matthew Parker Street, Westminster, S.W. The telegraphic address and telephone number remain unchanged.

Wm. Moss & Sons, Ltd., have asked us to announce that, owing to the large increase of work in the Reinforced Concrete Department, they have removed to larger offices. Their new address is 48, Bedford Row, London, W.C.

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